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Tests of Reinforced Concrete Pier Footings

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TESTS OF REINFORCED CONCRETE PIER FOOTINGS

BY

CHARLES HARRIS

AND

JAMES VERNE RICHARDS

THESIS

FOR THE

DEGREE OF BACHELORS OF SCIENCE

IN

ARCHITECTURAL ENGINEERING

COLLEGE OF ENGINEERING

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

CHARLES HARRIS and JAMES VERNE RICHARDS

ENTITLED Tests of Reinforced Concrete Pier Footings

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Architectural Engineering

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TESTS OF REINFORCED CONCRETE PIER FOOTINGS

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I. Introduction.

Reinforced concrete pier footings, which have now come into such general use in all classes of building construction, demand the investigation which they have long deserved. At present very little is known about the nature and kind of stresses produced in them under load, yet they are constantly being designed for use in places where failure would result in heavy losses of life and property.

At the present time there is very little reliable experimental data at hand. One series of tests, consisting of 32 specimens, was conducted during the winter and spring of 1909 by Messers N. H. Hill and E. F. Zahrobsky at the University of Illinois. The results of the tests are given in the thesis of the above-named men for the degree of B. S. in Architectural Engineering. Their tests showed some very interesting things, as they were the first tests of this character that have ever been made, but the results were not representative enough to warrant the drawing of any well defined conclusions.

In order to supplement the first series of tests this second series, including 28 specimens, has been undertaken. A greater uniformity was observed in the mix and arrangement of the reinforcement in the present series than in the series of 1909. The use of Ames deflectometers made it possible to study the action of the footings under load. The theory of reinforced concrete footings is still in its infancy and future tests may throw more light upon the results thus far obtained.

A comprehensive study of the results will involve the determination of the following: (1) the disposition of the pres-

sures and point of application of the resultant, (2) the planes of maximum stresses due to bending moment and shear, (3) some method of analyzing the stresses which would lead to a practical formula for design.

In order to obtain laboratory results which would be useful in practical design, an apparatus had to be designed which would very closely imitate the conditions present in actual service. It is thought that the bed of springs and method of applying the load described under testing apparatus, page 10, accomplished this purpose as well as any method that could be used in the laboratory. In fact, the springs give a more uniform distribution of pressures than is probably encountered in practice. As a uniform bearing is the usual assumption made in practical design, it is felt that the results are, perhaps, more uniform than if the tests had actually been made upon the soil. Especially is this true because the springs automatically adjust themselves to any unit bearing up to the maximum capacity of the machine.

While the results of the series of tests described in this thesis are interesting and instructive, they are too few in number establish anything very definitely. Acknowledgement is made to Mr. N. H. Hill and Mr. E. F. Zahrobsky for the use of some of their results in the tables of this thesis.

II. Theory and Formulae

A pier footing which is subjected the concentrated load on the pier from above and the uniform upward pressure of the soil from below, presents a problem for which no simple rational solution has yet been developed. Owing to the complex nature of the problem, no strictly rational solution will be attempted here; instead, the usual approximate method will be followed as far as thought to be consistent with the results of these tests.

In this method the pressure on the footing from below is assumed to be uniformly distributed over the bottom of the footing. One fourth of the footing is then considered as carrying one fourth of the total load, as shown in Fig. I. The resultant of the forces on the triangle under consideration is at the centroid of the triangle, i.e., at a point one third of the distance from the outer edge of the footing to the center. It is assumed that this portion of the footing acts as a cantilever beam and that the stresses due to the moment are a maximum in the plane of the face of the pier. The latter assumption is borne out by these tests, by the tests made in the same apparatus last year, and by some wall footing tests made at this university.

After making the above assumptions, it becomes necessary, in considering the reinforced footings, either to assume a given width for the above section or to assume a certain average stress in the reinforcement. When either of the last two values has been assumed, the other may be calculated. In these tests the problem has been investigated on the assumption that in all footings failing by tension in the steel, the steel was stressed to the

yield point when failure occurred. In the analysis it is thought advisable to make the following determinations: (1) width of footing effective in taking beam action, (2) relation of this width to some dimension of the footing, (3) unit bond stress developed, (4) unit shearing stress developed, and (5) the unit tensile stress developed in the steel in footings which failed in bond or shear.

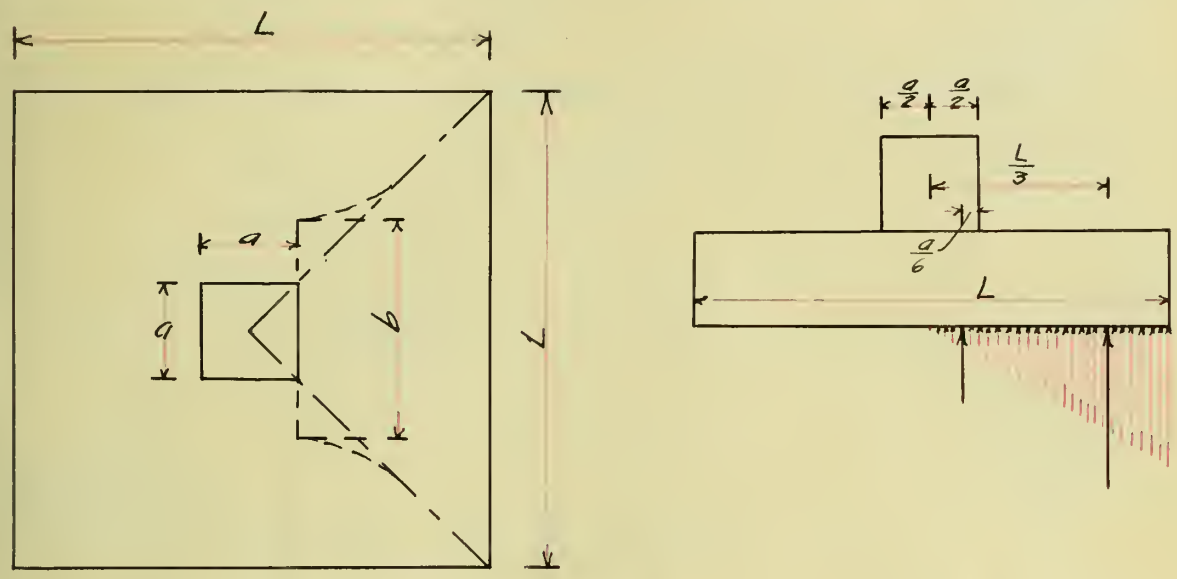


Fig. I.

Notation and Formulae

f_s = unit tensile stress in steel.

b = width footing effective in taking beam action.

d = distance from compressive face of the concrete to the center of the steel reinforcement.

M_b = bending moment on one quarter of the footing.

M_r = resisting moment in effective width.

W = total load on pier.

p = percentage of reinforcement.

L = length of one side of the footing.

a = length of one side of the pier.

U = unit bond stress.

V = total shear at any section.

v = average unit shearing stress.

Σo = summation of the perimeters of the steel rods in the width b.

S = average modulus of rupture determined from tests on control beams.

x = constant which when multiplied by the depth to the steel and added to the distance a, gives the effective width of the footing.

j = ratio of arm of resisting couple to d, taken equal to .875 in the following calculations.

From Fig. 1.,

$$M_b = \frac{W}{4}\left(\frac{L}{3} - \frac{a}{2}\right) - \frac{W}{4}\left(\frac{a^2}{L^2}\right)\left(\frac{a}{6}\right).$$

$$M_r = .875\ d \times f_s \times pbd.$$

$$= .875\ d^2 \times f_s \times pb.$$

Whence, $b = \dfrac{M_b}{.875\ pf_s d^2}$. - - - - - - - - - - - - (1).

And $f_s = \dfrac{M_b}{.875\ pbd^2}$ - - - - - - - - - - - - (2).

For plain concrete footings,

$$M_b = M_r = \frac{S\ I}{c}.$$

$$\frac{bd^3}{12} = \frac{Mc}{S}, \quad \text{but } c = \frac{d}{2}$$

Therefore, $\dfrac{bd^2}{12} = \dfrac{M}{S}$ and $b = \dfrac{6M}{Sd^2}$. - - - - - - - (3).

In the following tables the effective widths, b, given for footings failing by tension in the steel were computed from equation (1), assuming that the failure occurred at the yield point of the steel. The value used for the yield point in these calculations was carefully determined from 43 tests upon rods of the same size and same shipment. The effective widths assumed for the footings which failed in shear, diagonal tension, or bond, was the average of the values computed for the footings which failed in tension in the steel.

Bond stresses were calculated for the same effective widths as were determined by the above methods. The following formula was used: $U = \frac{V}{\Sigma o \times jd}$. Turneaure and Maurer, "Principles of Reinforced Concrete Construction".

The shearing stresses were calculated upon the assumption that each quarter of the load upon the pier is distributed uniformly over a shearing area equal to (a + d)d, as shown in Fig. 2.

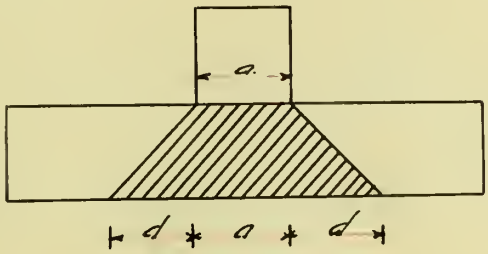


Fig. 2.

If $V = \frac{W}{4}$ on the shaded section,

Then $v = \frac{V}{(a + d)\ d}$.

The above assumption is not strictly rational since there must be diagonal tension entering into the type of failures described in the tests as shear failures. The actual plane of

cleavage determined in the tests is inclined at an angle of about 45 degrees with the axis of the pier, due to the action of the diagonal tension. It is thought allowable, however, to base the strength of footings failing in this manner upon the shearing stresses in the vertical plane above described.

III. Materials, Test Pieces, Apparatus, and Method of Testing.

The materials used were the same as are used in first class reinforced concrete construction. The cement and steel were furnished by the manufacturers. The sand and stone were purchased in the open market.

Stone: The stone used was a good quality of hard limestone from Kankakee, Illinois, ordered screened through a 1-in. screen and over a 1/4-in. screen.

Sand: The sand used was a clean sharp sand from a bank near the Wabash River at Attica, Indiana. It contained about 47% voids and weighed about 102 lb. per cu. ft.

Cement: The Universal Portland Cement Co. furnished the cement used in the tests. Table I. gives the average tensile strength of the cement at 7 and 28 days.

Tensile Tests of Universal Portland Cement.

(Each value is the average of 5 briquets)

Sample	7 days		28 days	
	Neat	1-3	Neat	1-3
1	670	190	697	297
2	629	219	678	328
3	613	217	649	315

Concrete: The concrete was thoroughly mixed by hand on the floor of the laboratory by men accustomed to the preparation of concrete test specimens. Enough concrete was mixed in a batch to make one footing, one control beam, 6 by 8 in. by 3 ft., and three 6 in. cubes. The cement and sand were first mixed dry. Stone was then added and thoroughly mixed, enough water

being used to make a fairly wet concrete. The mixture was in all cases 1 - 2 1/2 - 5 by loose volume, 95 lb. of cement being considered a cubic foot.

Steel: The steel was ordinary commercial steel having an average yield point, for 43 tests on 1/2 in. rods, of 38100 lb.per sq. in. None of the footings having 3/4 in. rods failed by tension in the steel, hence the yield point of this size was not needed in the calculations and was not determined. The corrugated bars were all of the same cross sectional area, 9/16 - in. rounds and 1/2-in. squares, furnished by the Corrugated Bar Company of St. Louis. They were all high-carbon steel having an average yield point of 55300 lb. per sq. in.

Test Pieces: The footings tested in this series were all 5 ft. square and varied from 5 to 10 in. deep to the steel in the reinforced footings and from 6 to 18 in. over all in the plain concrete footings. They were made directly on the concrete floor of the laboratory, a strip of building paper preventing any bonding of the footing to the floor. The forms were allowed to remain in place one week and the completed footing stored in any convenient place in the laboratory. The control beam and three cubes made from each batch of concrete were for the purpose of obtaining the modulus of rupture and compressive strength of the concrete from each batch.

The footings numbered 28 in all and were made in pairs, corresponding specimens being made at different times during the period of manufacture. The first footing was made on December 20, 1909, and the last one on February 18, 1910. There were eight plain footings numbered from 1501 to 1508 and twenty reinforced footings numbered from 1515 to 1564.

It was originally intended to test all of the footings at an age of about 60 days, but owing to a delay in some slab tests that required the use of the testing apparatus, a few of the footings were over 80 days old when tested.

Testing Apparatus: The apparatus used was specially designed and built for a similar series of tests during the winter and spring of 1909. The pressure on the pier was supplied by means of two 100-ton hydraulic jacks mounted upon a 24-in. I-beam and tied to the foundation by means of 8 rods 1 1/2 in. in diameter. A bed of 113 car springs, ground to a uniform length of 12 in., distributed the load uniformly over the bottom of the footing. The springs rested upon a base of 10-in. by 30-lb. I-beams placed flange to flange. The capacity of the apparatus was limited by the springs which could sustain a maximum load of about one ton each. 112 additional springs could be inserted in the spaces between the others in case more load was required. Using the full number of springs the capacity of the apparatus might be increased to about 225 tons. The photographs on the last few pages of this thesis show details of the testing apparatus.

Method of Testing: The tests required the services of three men, one to operate the oil pumps that supplied the hydraulic jacks, one as observer and the other to map cracks, record deflections, and keep the test notes. The load was applied in varying increments for the footings of the different kinds, as will be found by reference to the load-deflection graphs; but was kept as nearly as possible equally divided between the jacks. Considerable trouble was experienced in some of

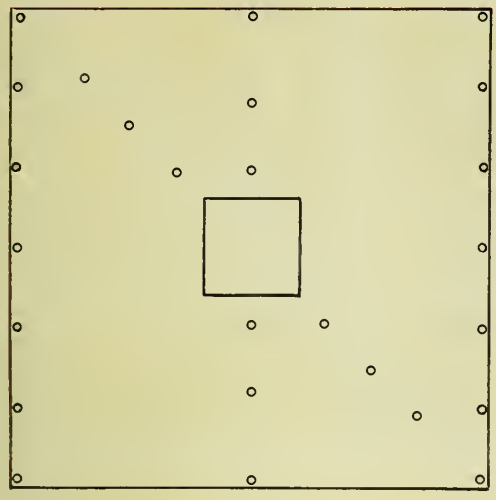


Fig. 3.

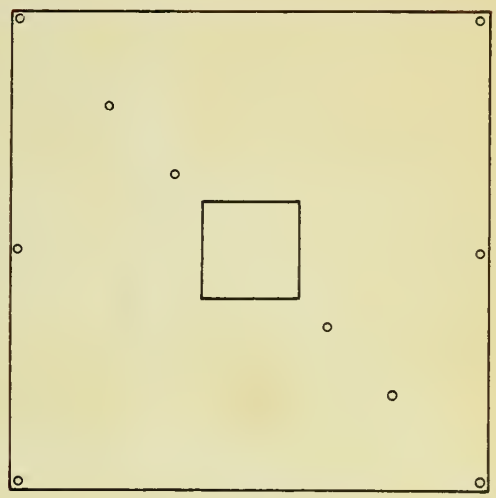


Fig. 4.

the tests with the apparatus. The I-beam on which the jacks were mounted would sometimes get badly out of plumb, due to unequal tension in the side rods, and usually at the same time the springs would lean dangerously in one direction. Except in cases where such trouble was experienced, the total load was not released until the ultimate failure of the footing.

The deflections were obtained by means of Ames deflectometers attached to a framework of 1 1/4 by 1 1/4-in. angles which was supported at three points on the upper surface of the footing close to the pier. The general arrangement of the instruments is shown in Figs. 3 and 4 and by the photograph on page 95. The instruments were arranged according to Fig. 3 on footings No. 1505 and 1531. All others were according to Fig. 4. Defections could not be taken on the 6-in.-thick plain footings because of the small loads at which they failed. The instruments indicated automatically on a dial the movements of the various parts of the upper surface of the footing with respect to a fixed section at the pier. Deflections could be read accurately to the

third decimal place and could be estimated to the fourth decimal place. The instruments were always removed before the ultimate load was reached in order to avoid any danger to them.

The closure of the springs at the four corners was measured in most of the tests at intervals during the loading to determine whether or not the footing settled equally at those points. These readings were taken only as a check and were not used in any of the calculations or tables.

Table 2.

Data of Footings

No.	Kind of concrete by weight.	Depth over all in.	Depth to steel in.	Percent of reinforcement.	Disposition of reinforcement.
1501	1-2.60-4.43	6	-	-	None
1502	1-2.08-3.62	6	-	-	None
1503	1-2.62-4.50	8	-	-	None
1504	1-2.67-4.38	8	-	-	None
1505	1-2.72-4.53	12	-	-	None
1506	1-2.61-4.66	12	-	-	None
1507	1-2.60-4.63	18	-	-	None
1508	1-2.63-4.48	18	-	-	None
1515	1-2.61-4.58	12	10	0.4	12 1/2in. rounds 5in. c-c,
1516	1-2.51-4.66	12	10	0.4	each way.
1521	1-2.61-4.59	8	7	0.58	do.
1522	1-2.56-4.60	8	7	0.58	
1525	1-2.56-4.56	6	5	0.8	do.
1526	1-2.92-4.50	6	5	0.8	
1531	1-2.62-4.49	12	10	0.6	18 1/2in. rounds 3 in. c-c,
1532	1-2.57-4.58	12	10	0.6	each way.
1535	1-2.68-4.66	12	10	0.6	8 3/4in. rounds 8in. c-c,
1536	1-2.58-4.60	12	10	0.6	each way.
1541	1-2.60-4.51	6	5	1.2	do.
1542	1-2.60-4.48	6	5	1.2	
1551	1-2.61-4.43	12	10	0.4	10 9/16in. corr. rounds
1552	1-2.64-4.56	12	10	0.4	6in. c-c, each way.
1553	1-2.62-4.45	12	10	0.6	15 9/16in. corr. rounds
1554	1-2.59-4.42	12	10	0.6	4in. c-c each way.
1561	1-2.61-4.39	12	10	0.6	1/2in. rounds 7in. c-c,
1562	1-2.60-4.68	12	10	0.6	uniformly in four directions.
1563	1-2.62-4.49	12	10	0.6	Same as 1561, except omit
1564	1-2.64-4.66	12	10	0.6	3 short diagonal rods in corners.

All footings were 1-2 1/2- 5 by loose volume of materials.

Table 3.

Calculated Data of Plain Footings

No.	Age days	Depth inches	Maximum load pounds	Modulus of rupture	b inches	x where b = a+xd.
1501	77	6	39000	309	63.6	8.60
1502	74	6	36000	423	58.7	7.78
1503	77	8	60000	372	55.1	5.39
1504	73	8	40000	425	36.7	3.09
1505	86	12	100000	376	40.8	2.40
1506	77	12	80000	272	32.6	1.72
1507	75	18	256000	409	46.5	1.92
1508	73	18	212000	363	38.5	1.47

Note: The quantities in the last two columns were calculated from the flexural equation, $S = \frac{M\ c}{I}$, in which S is the average of all the moduli of rupture obtained from the control beams, 356 lb. per sq. in.

Table 4.

Test Data of Reinforced Footings.

No.	Load at first visible crack lb.	Maximum load lb.	Manner of Failure.
1515	140 000	208 000	Gradual tension followed by shear.
1516	120 000	192 000	do.
1521	64 000	132 000	do.
1522	100 000	142 000	do.
1525	56 000	76 000	Gradual tension and bond.
1526	48 000	100 000	do.
1531	200 000	300 000	Shear or diagonal tension.
1532	140 000	252 000	do.
1535	180 000	216 000	Bond.
1536	140 000	204 000	do.
1541	56 000	64 000	do.
1542	56 000	112 000	do.
1551	130 000	246 000	Shear or diagonal tension.
1552	120 000	156 000	do.
1553	180 000	346 000	do.
1554	140 000	308 000	do.
1561	160 000	260 000	Gradual tension followed by shear.
1562	140 000	232 000	do.
1563	140 000	196 000	do.
1564	160 000	232 000	do.

Table 5.

Calculated Data of Reinforced Footings

No.	Depth in.	Tensile stress in steel lb/sq in.	Bond lb/sq in.	Shear lb/sq in.	b in.	x where b = a+xd	Manner of failure
1515	10	38100	347	359	54.5	4.25	Tension
1516	10	38100	346	331	50.5	3.85	do.
1521	7	38100	352	342	49.0	5.28	do.
1522	7	38100	361	369	52.5	5.72	do.
1525	5	38100	342	287	40.0	5.60	do.
1526	5	38100	347	378	52.5	8.01	do.
1531	10	47400	455	518	42.0	3.0	Shear
1532	10	39800	382	435	42.0	3.0	do.
1535	10	34100	500	373	42.0	3.0	Bond
1536	10	32200	473	352	42.0	3.0	do.
1541	5	20200	312	242	40.0	6.0	do.
1542	5	35400	545	424	40.0	6.0	do.
1551	10	58200	570	425	42.0	3.0	Shear
1552	10	37000	317	269	42.0	3.0	do.
1553	10	54600	490	596	42.0	3.0	do.
1554	10	43000	437	531	42.0	3.0	do.
1561	10	38100	Diagonal rods	448	45.7	3.37	Tension
1562	10	38100		400	40.7	2.87	do.
1563	10	38100		338	34.3	2.23	do.
1564	10	38100		400	40.7	2.87	do.
1413*	10	38100	349	250	37.6	2.57	do.
1414*	10	38100	387	330	31.3	1.93	do.
1415*	10	38100	415	276	40.7	2.87	do.
1416*	10	38100	272	220	33.0	2.10	do.

* Taken from thesis of N. H. Hill and E. F. Zahrobsky, 1909.

For all footings failing by tension the stress in the steel is assumed to be at the yield point, 38100 lb. per sq. in., and the other values obtained from that. The average x for 10-in. footings is 3.0 , and for 5-in. footings is 6.0 . In all footings failing by shear or bond the effective width, b , was assumed to be the same as for footings of same depth failing by tension and the other values in the table calculated from this.

Table 6.

Tests of Six-inch Cubes.

(In each case the age and unit stress given are the averages of three cubes)

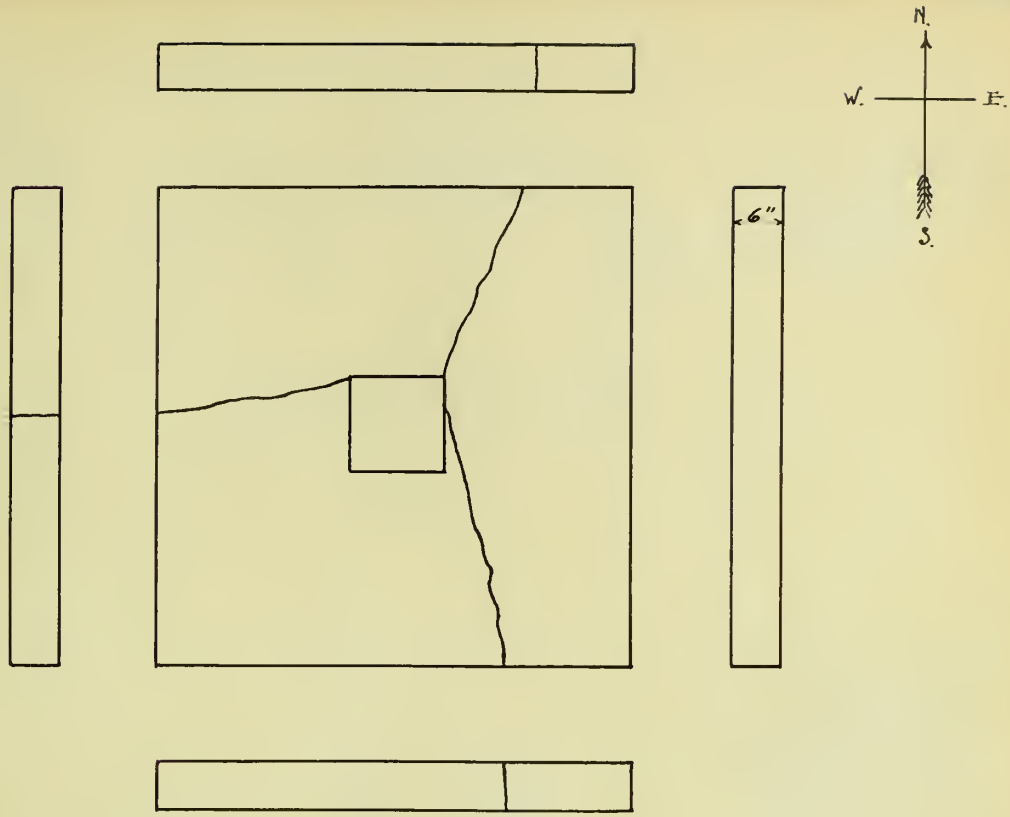
No.	Age days	Unit compressive strength. lb. per sq. in.
1501	112	2910
1502	103	2939
1503	112	2307
1504	109	1761
1505	128	3618
1506	115	2260
1507	121	3180
1508	109	1988
1515	127	2980
1516	120	1380
1521		
1522	94	2935
1525		
1526	90	1917
1531	120	3193
1532	114	2637
1535	121	2510
1536	94	2768
1541	109	1761
1542	78	2272
1551	121	2290
1552	101	2857
1553	120	2835
1554	103	2242
1561	114	2550
1562	87	2187
1563	113	2462
1564	83	2322

Table 7.

Tests of Control Beams.

(Beams were all 6 in. by 8 in. by 3 ft. loaded at the third points).

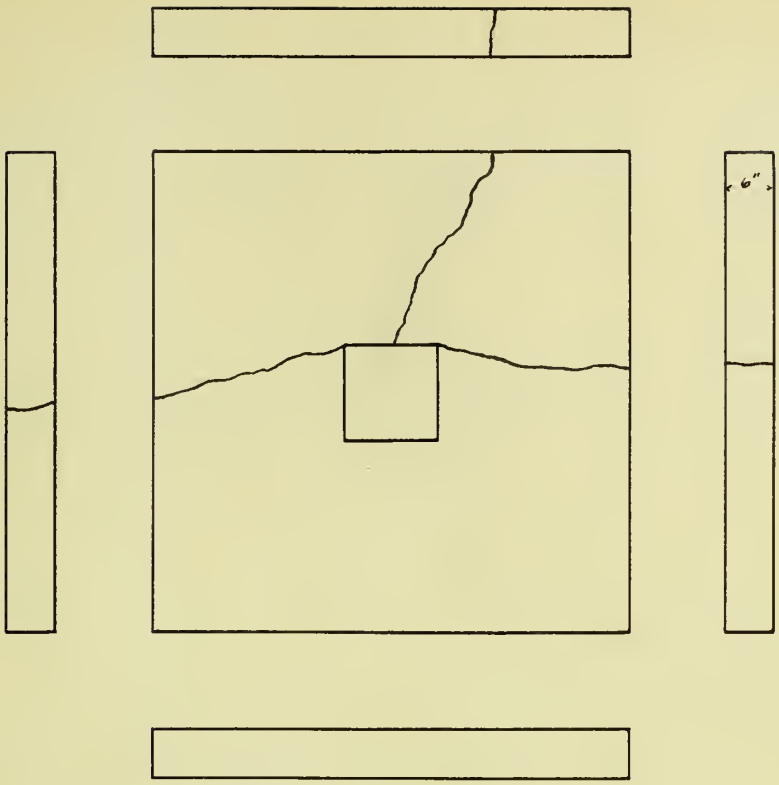
No.	Age days	Modulus of rupture. lb. / sq. in.
1501	79	309
1502	87	423
1503	79	373
1504	109	425
1505	95	376
1506	99	272
1507	105	409
1508	93	363
1515	95	371
1516	99	385
1521	107	357
1522	91	296
1525	107	294
1526	93	287
1531	93	342
1532	96	440
1535	105	377
1536	94	230
1541	104	387
1542	85	397
1551	88	363
1552	96	368
1553	87	365
1554	99	362
1561	81	375
1562	84	408
1563	80	306
1564	89	313
Averages	93.5	356



Footing No. 1501 --- Plain concrete 6 in. thick.

No deflections were taken in this test because of the danger to the instruments in case of an unexpected failure.

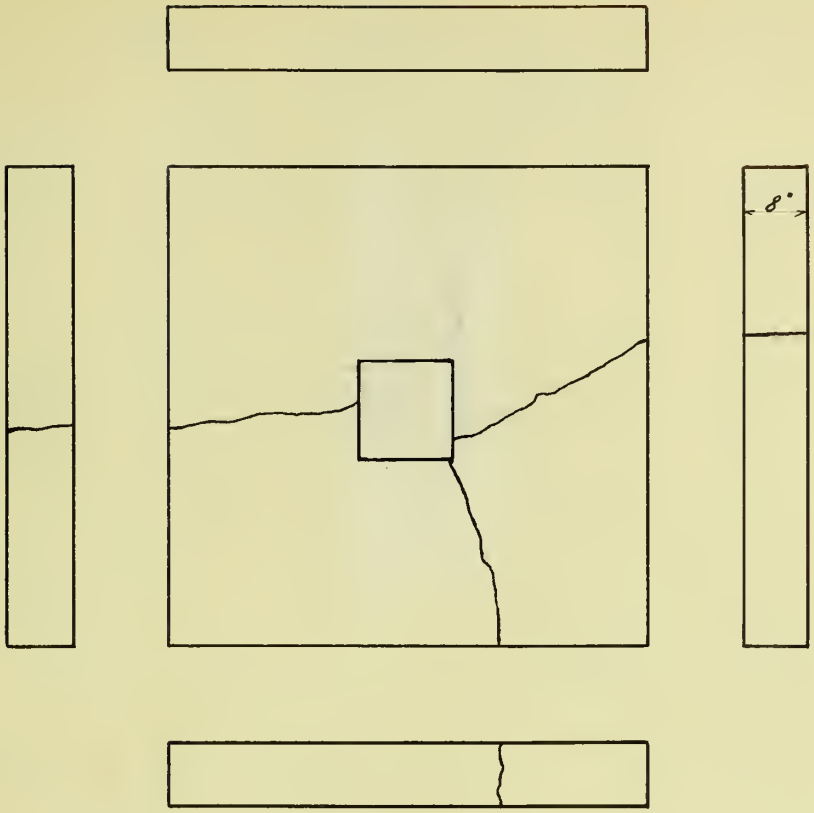
Failure occurred suddenly at 19.5 T. along the lines shown on the diagram.



Footing No. 1502 --- Plain concrete 6 in. thick.

No deflections were taken in this test because of the danger to the instruments in case of an unexpected failure.

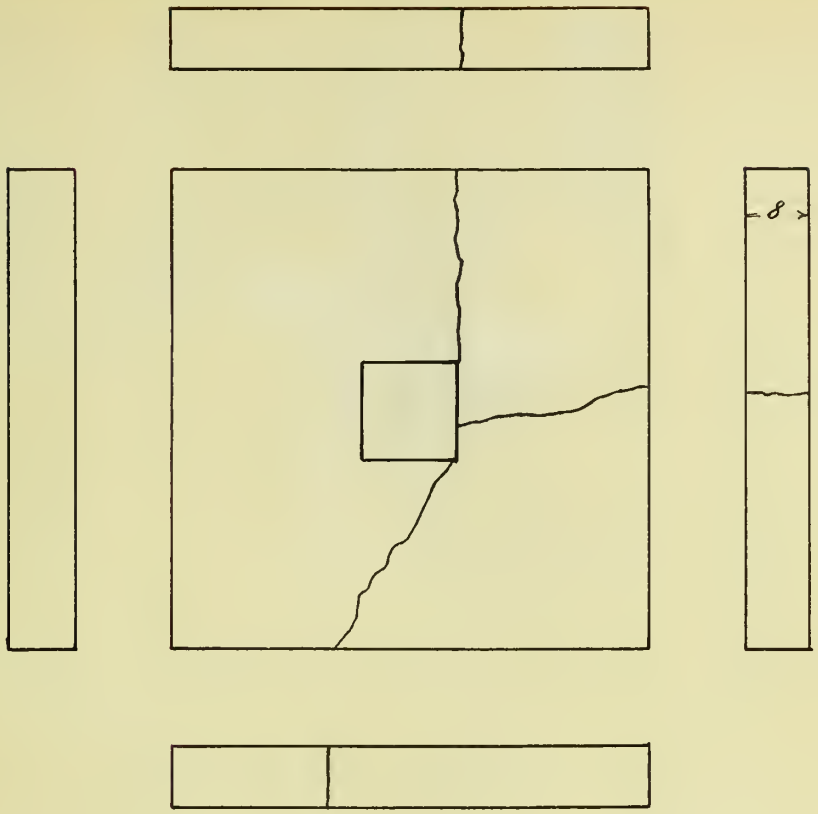
Sudden but not violent failure at 18 T.



Footing No. 1503 --- Plain concrete 8 in. thick.

No deflections were taken in this test because of the danger to the instruments in case of an unexpected failure.

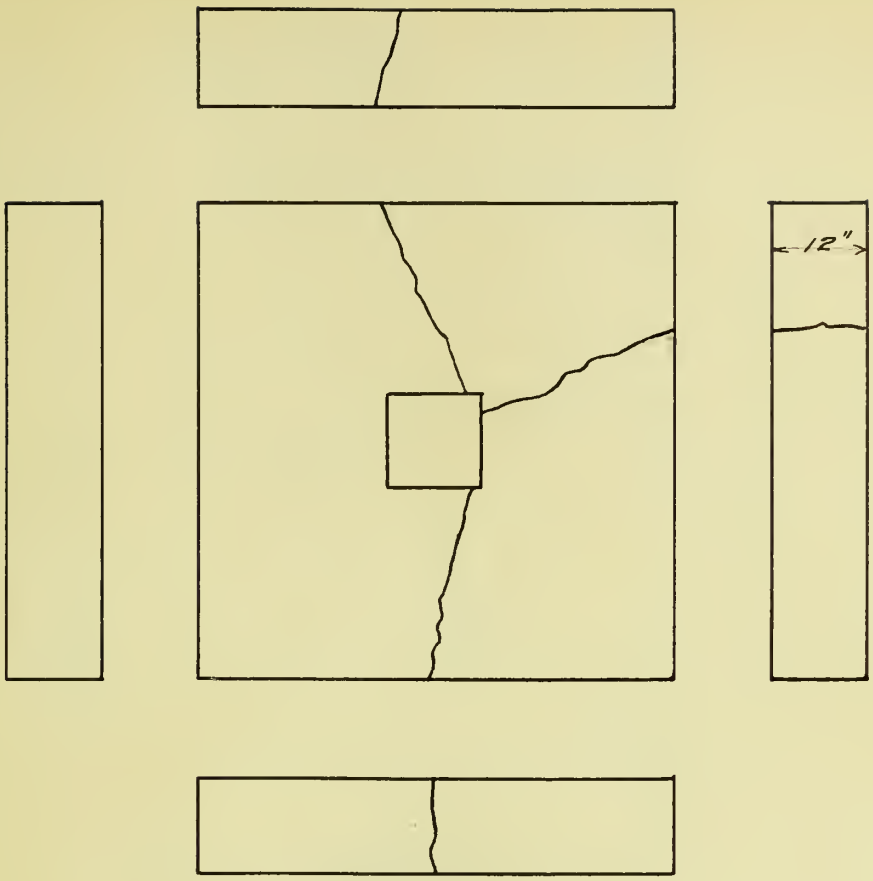
Failure occurred suddenly along the lines shown in the diagram at 30 T.



Footing No. 1504 --- Plain concrete 8 in. thick.

No deflections were taken in this test because of the danger to the instruments in case of an unexpected failure.

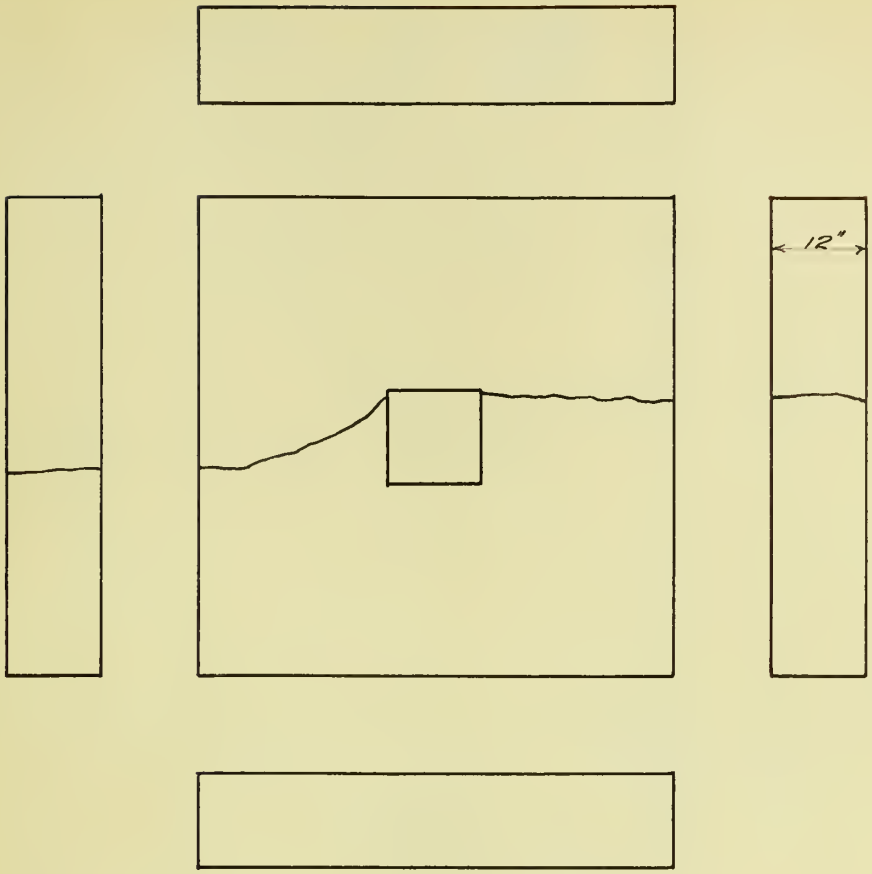
Failure at 20 T. was sudden but without loud report as in the case of the other plain footings.



Footing No. 1505 --- Plain concrete 12 in. thick.

Sudden and violent failure at 50 T. before the deflectometers had been removed from the frame.

The jar damaged several of the deflectometers, kicked the entire spring foundation out of place and the S. E. corner clear off onto the floor.

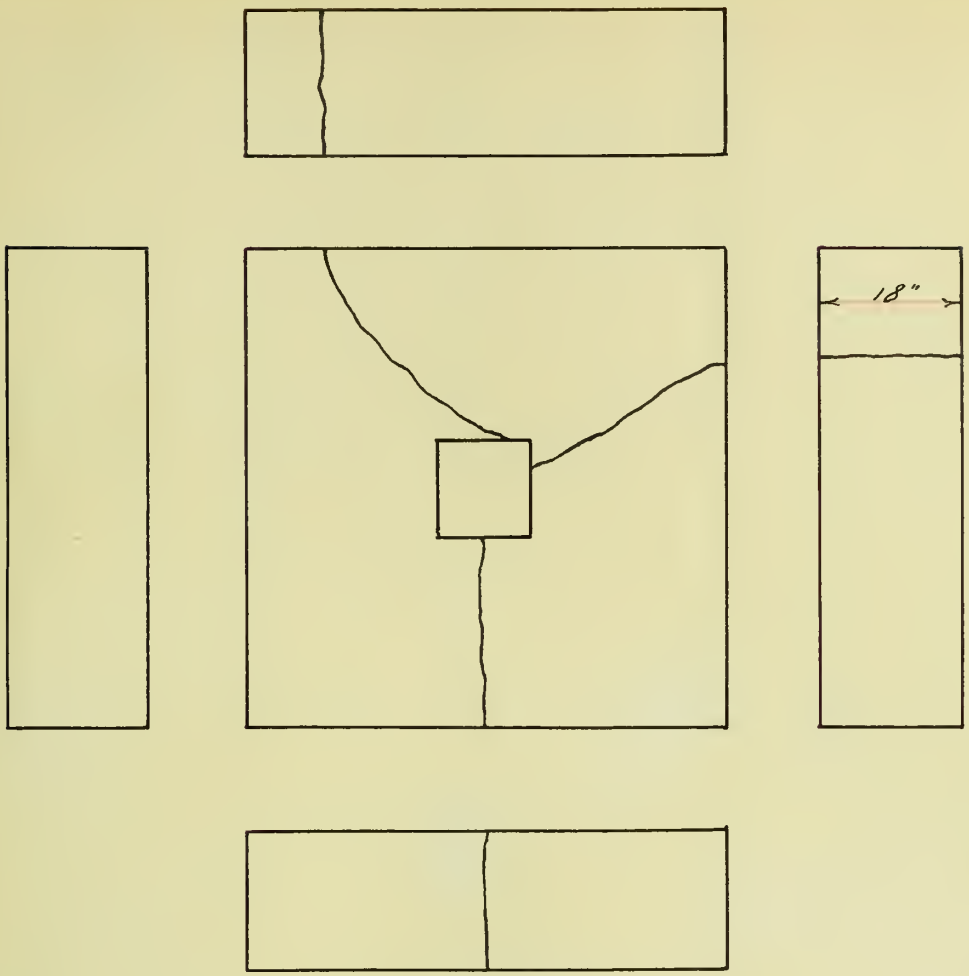


Footing No. 1506 --- Plain concrete 12 in. thick.

The instruments were removed at 36 T.

Failure occurred at 40 T. suddenly but without much jar.

The maximum load was held for about a minute before failure.



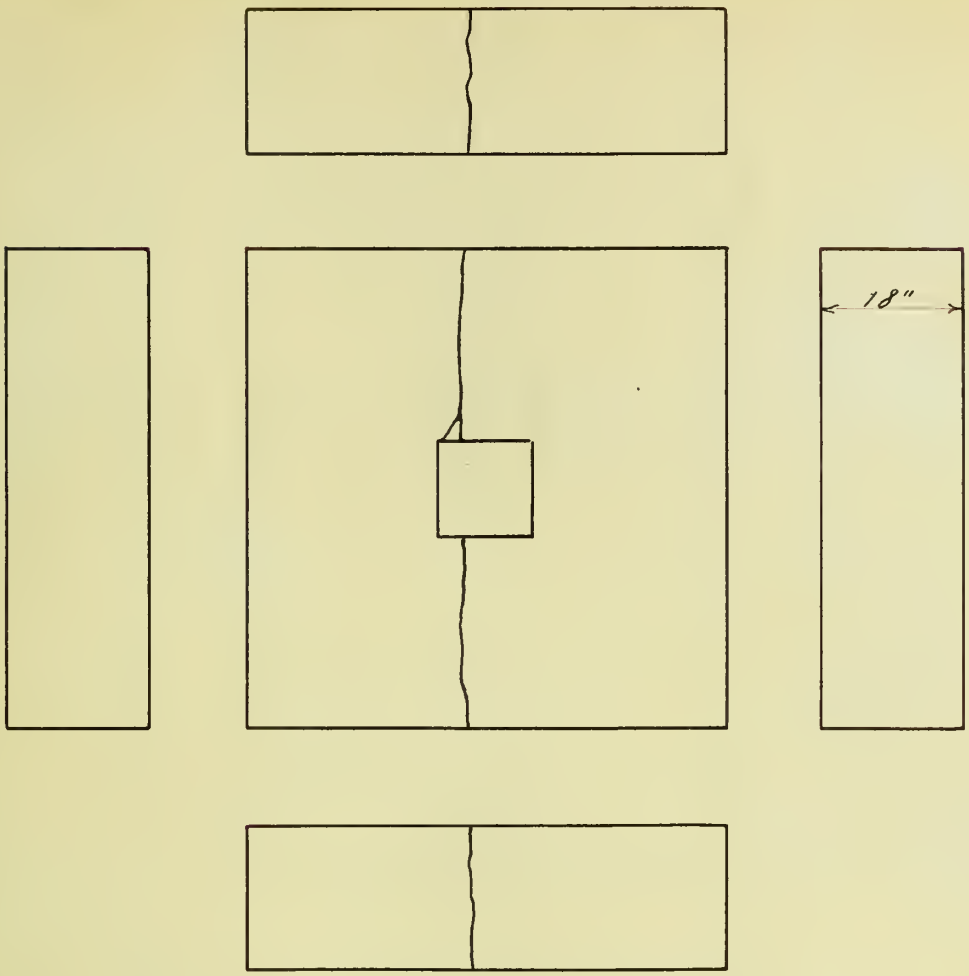
Footing No. 1507 --- Plain concrete 18 in. thick.

The instruments were removed at 60 T. with no indications of failure.

At 128 T. the footing broke violently into three pieces as shown in diagram.

The pier was separated entirely from the other three pieces except for the four steel rods which run down the corners. These were badly bent and twisted.

See photograph on page 91.



Footing No. 1508 --- Plain concrete 18 in. thick.

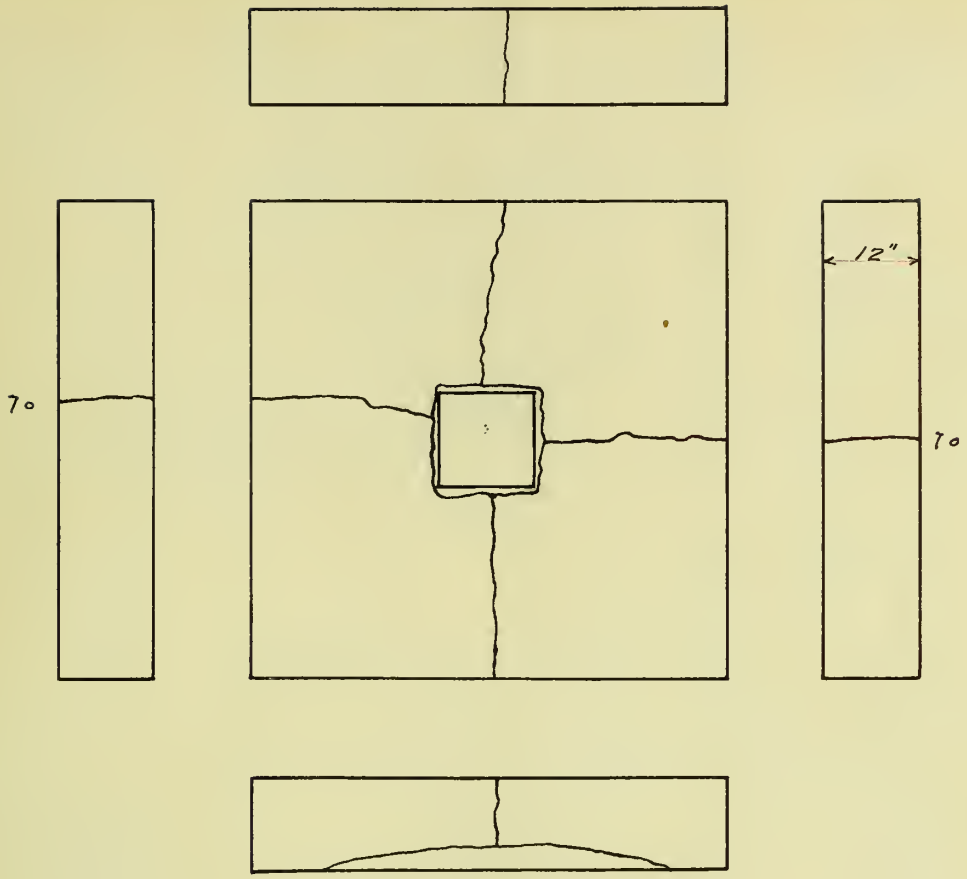
Instruments removed at 60 T.

At 104 T. the springs were so badly out of line that the load was slacked off. No indications of failure thus far.

Loaded again the next day on same bed of springs.

Sudden and disastrous failure at 106 T. The machine was almost a total wreck and the steel frame upon which the deflectometers are mounted was badly bent.

See photograph on page 90.



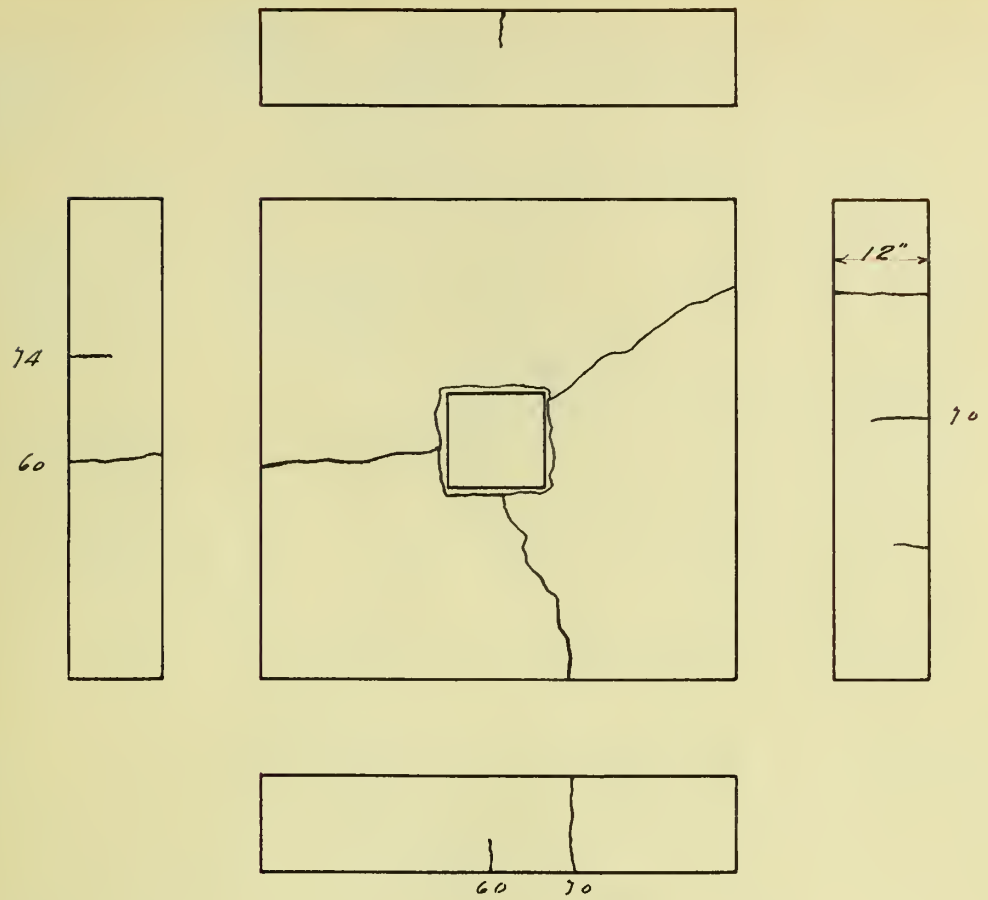
Footing No. 1515 --- 0.4% each way 10 in. deep.

The first cracks appeared at 70 T., one on the W. opposite the N. edge of the pier and one on the E. about the middle.

At 94 T. cracks appeared on N. and S. faces near the center.

At 104 T. gradual failure in tension followed by sudden shearing through of pier as shown in sketch.

Ropes and chains were required to remove the footing from the machine.



Footing No. 1516 --- 0.4% each way 10 in. deep.

First cracks at 60 T. as shown.

Instruments removed at 80 T.

Load released at 84 T. to plumb the upper beam.

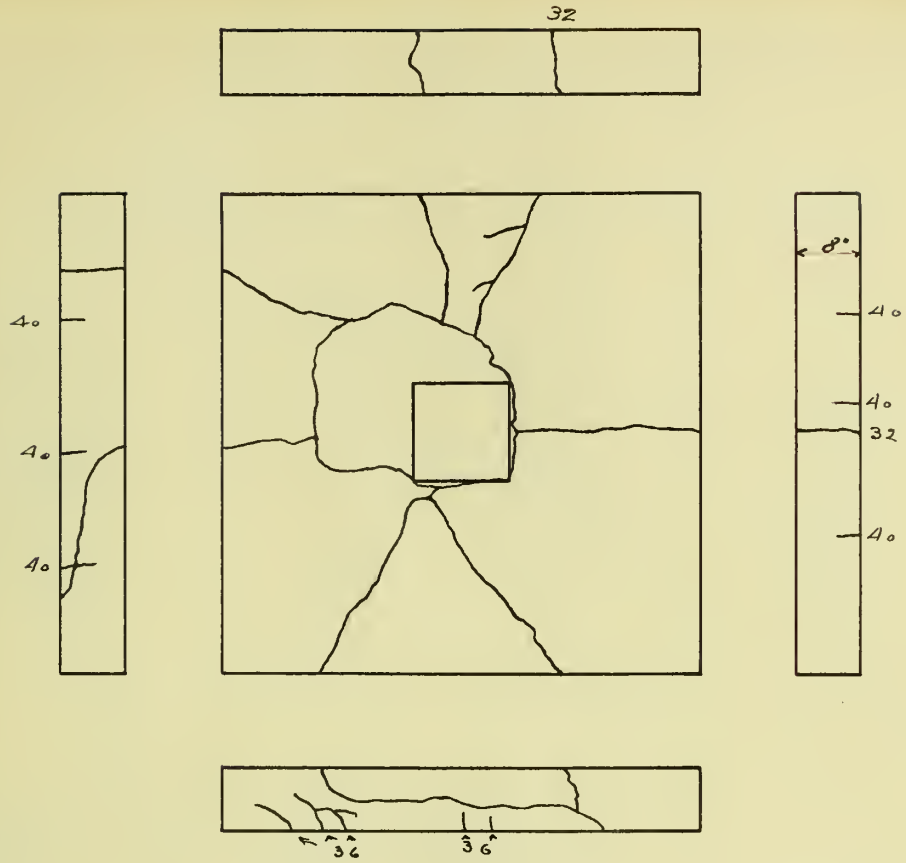
Load again brought to 80 T. and released to adjust upper nuts to an even bearing.

Load again applied and released because of dangerous leaning of springs. Left until following day.

The footing held the maximum load of 96 T. for several minutes under steady pumping while cracks on the sides slowly opened.

Final failure was by slipping of rods as indicated by the opening of a large crack on the E. near the N. face at the maximum load. The pier also sheared through.

A portion of the concrete fell away from the S. face at failure exposing the ends of some of the rods. The ends had slipped about an inch from their original position.

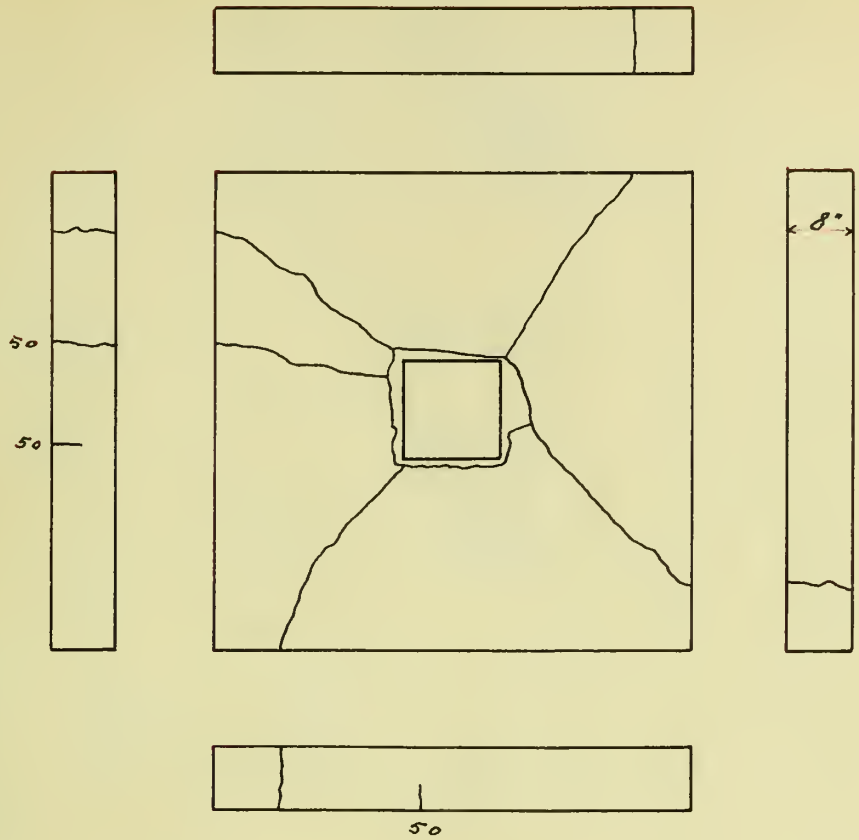


Footing No. 1521 --- 0.58% each way 7 in. deep.

First crack at 32 T.

At 68 T. the cracks widened perceptibly and load fell off to 66 T. The footing failed at this load by a gradual opening of the cracks followed by a shearing through of the pier as shown in the diagram.

See photgraph on page 92.

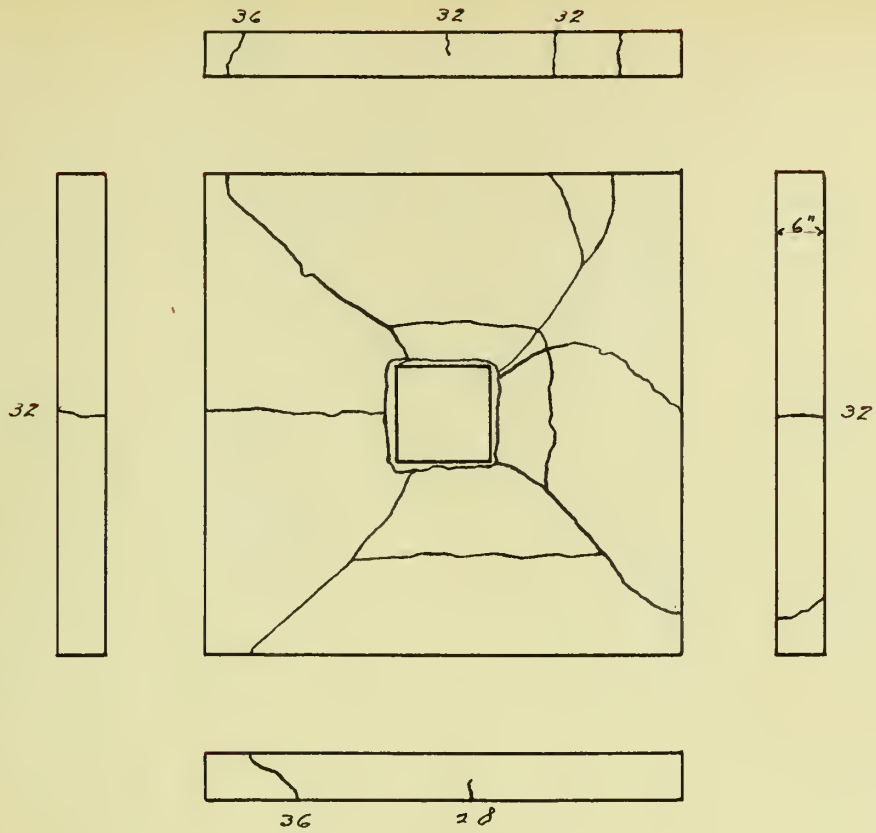


Footing No. 1522 --- 0.58% each way 7 in.deep.

First cracks at 50 T.

Instruments removed at 50 T.

Very gradual failure at 71 T. by opening of cracks across face as shown and slight punching in of pier.



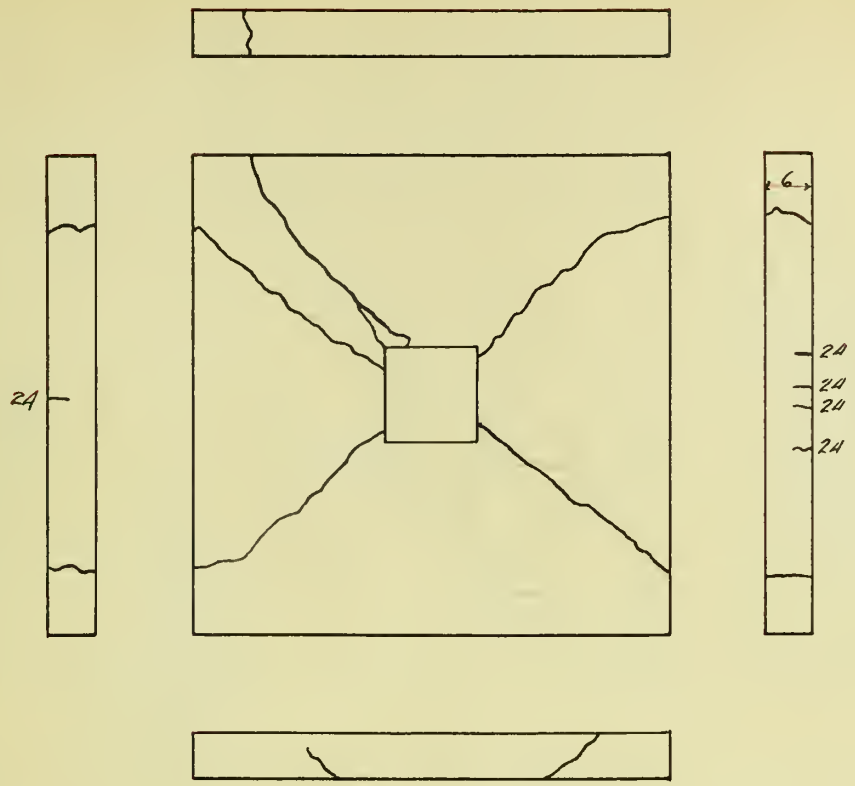
Footing No. 1525 --- 0.8% each way 5 in. deep.

First crack at 28 T. and others as shown on the diagram.

The load had to be released at 34 T. because of the failure of some wooden blocking on top of the pier.

Load was again applied without the instruments until failure at 38 T.

Failure was very gradual by the opening of the cracks. The cracks on the diagonals opened the widest. Finally the pier sheared through. An examination of the bottom of this test piece showed that the shearing in this particular case was nearly vertical.



Footing No. 1526 --- 0.8% each way 5 in. deep.

First cracks at 24 T.

Instruments removed at 30 T.

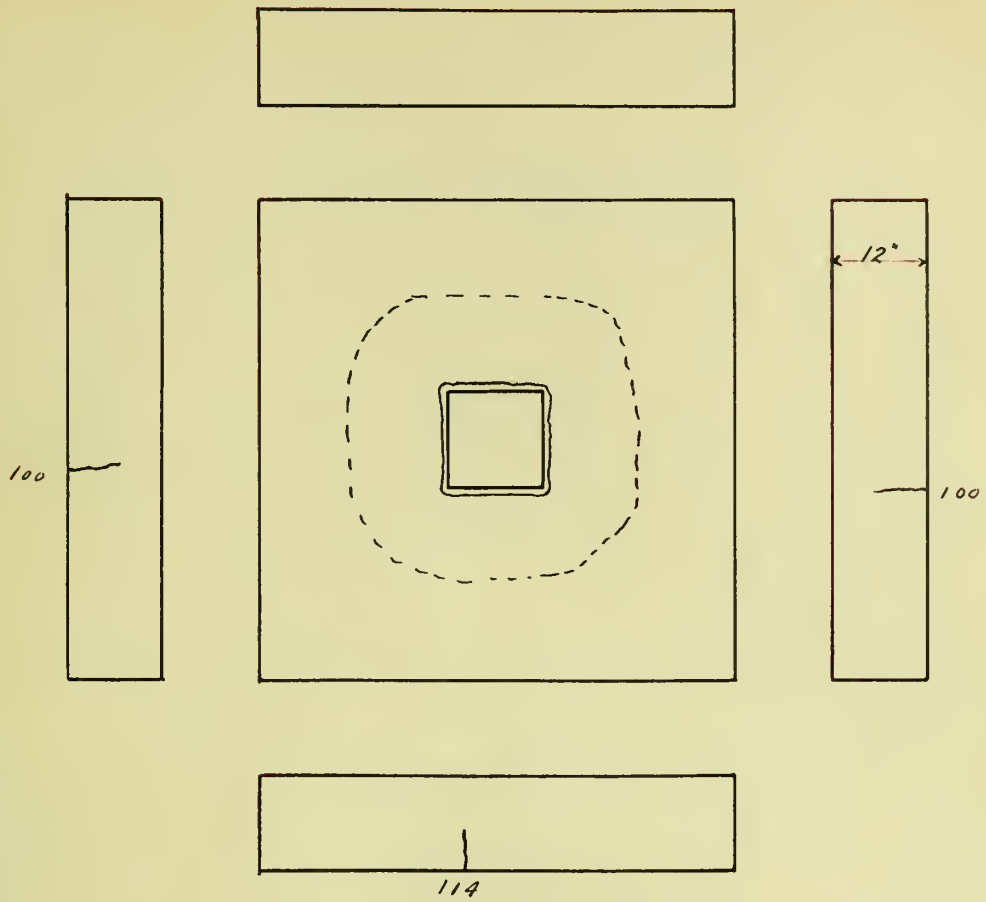
At 50 T. the cracks on faces near the corners opened wide

and load began to fall off under steady pumping.

Final failure occurred by cracking across top of footing as shown

This, like 1541, was an extremely gradual failure.

This type of failure would ordinarily indicate a failure in the bond between the steel and the concrete.



Footing No. 1531 --- 0.6% each way 10 in. deep.

At 100 T. the springs leaned dangerously so jacks were released. The first hair cracks appeared at this load.

The bearing block was then shifted slightly and the load again applied. Same trouble at 100 T. so jacks were again released and footing put aside until more springs could be inserted.

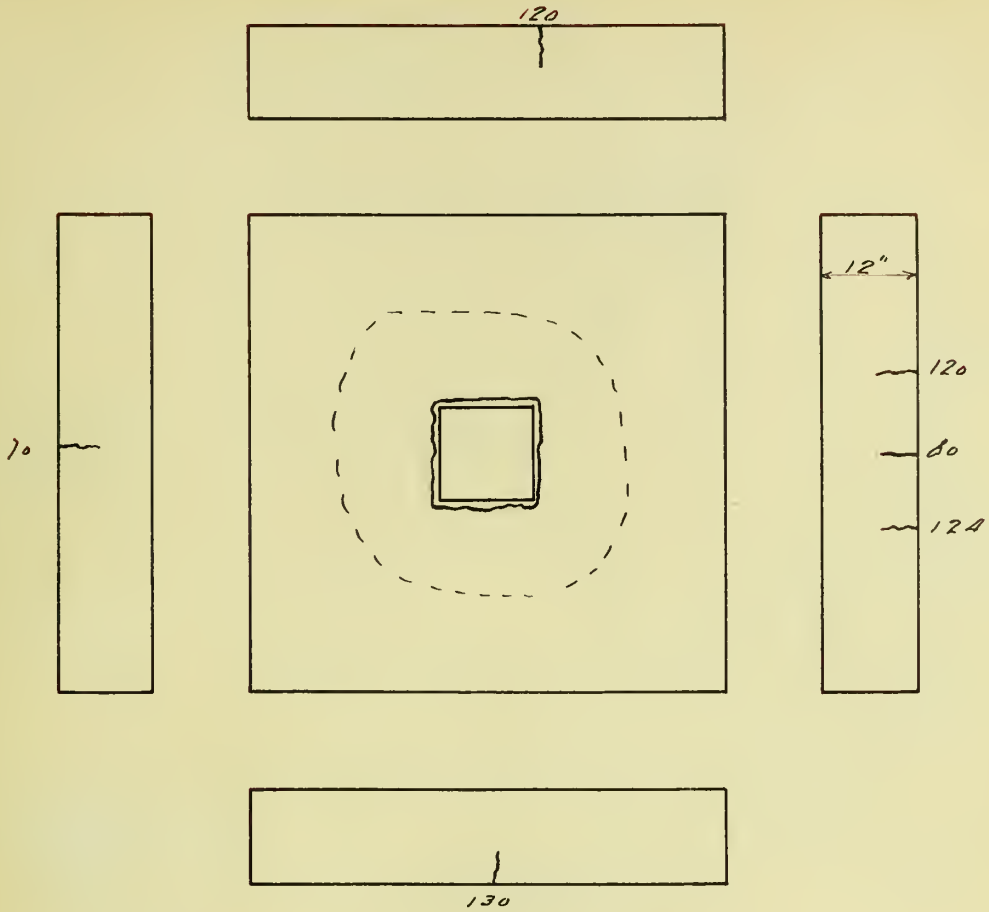
Footing was again loaded four weeks later with the remainder of the springs in place.

At 150 T. the concrete about the base of the pier began to crumble. Pumping was stopped and the load began to fall off slowly.

At 146 T. the pier sheared through,loosening the two inches of concrete below the steel. This was not accompanied by any marked opening of the cracks in the faces so the failure was a shear failure.

The pier had to be clamped to the footing in order to be removed from the machine with it.

See photograph on page 96.



Footing No. 1532 --- 0.6% each way 10 in. deep.

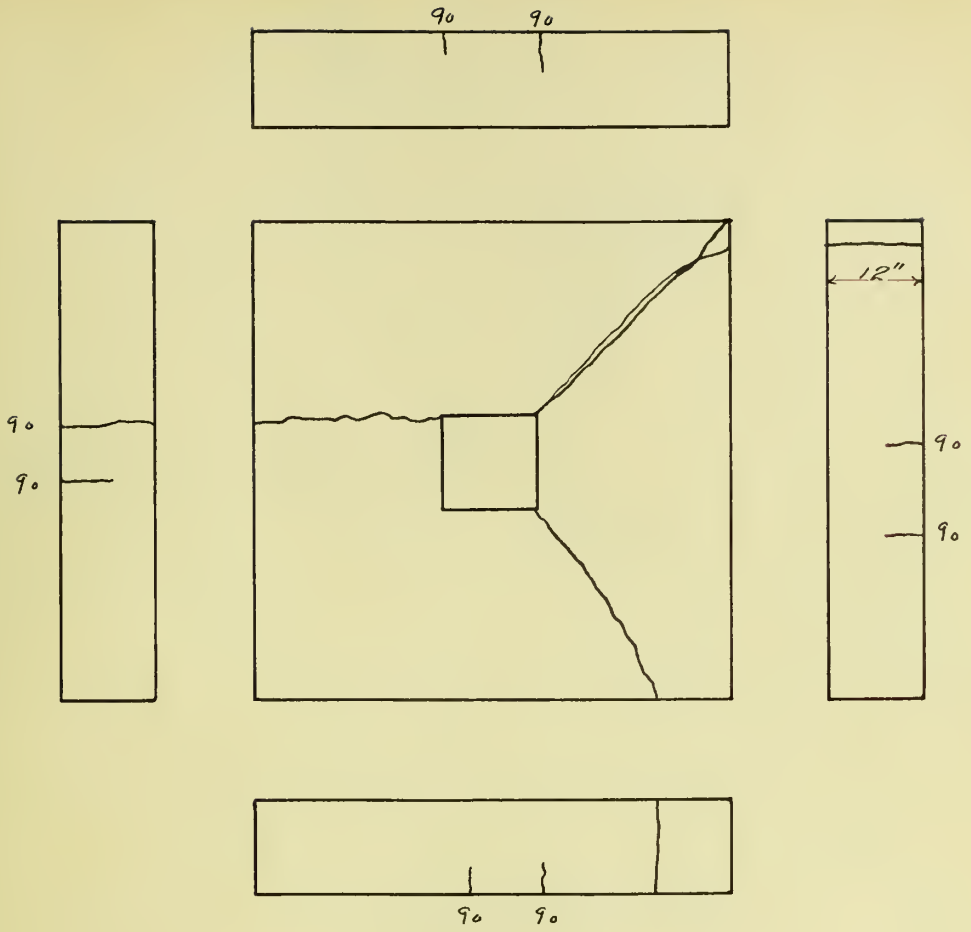
First crack in middle of E. face at 80 T.

Instruments removed at 110 T.

At 131 T. the springs had closed so load was released. The small hair cracks shown on the diagram were the only indications of failure thus far.

Loaded to failure at 136 T. one week later, after the remainder of springs had been placed.

Failure was sudden by shearing through of pier.



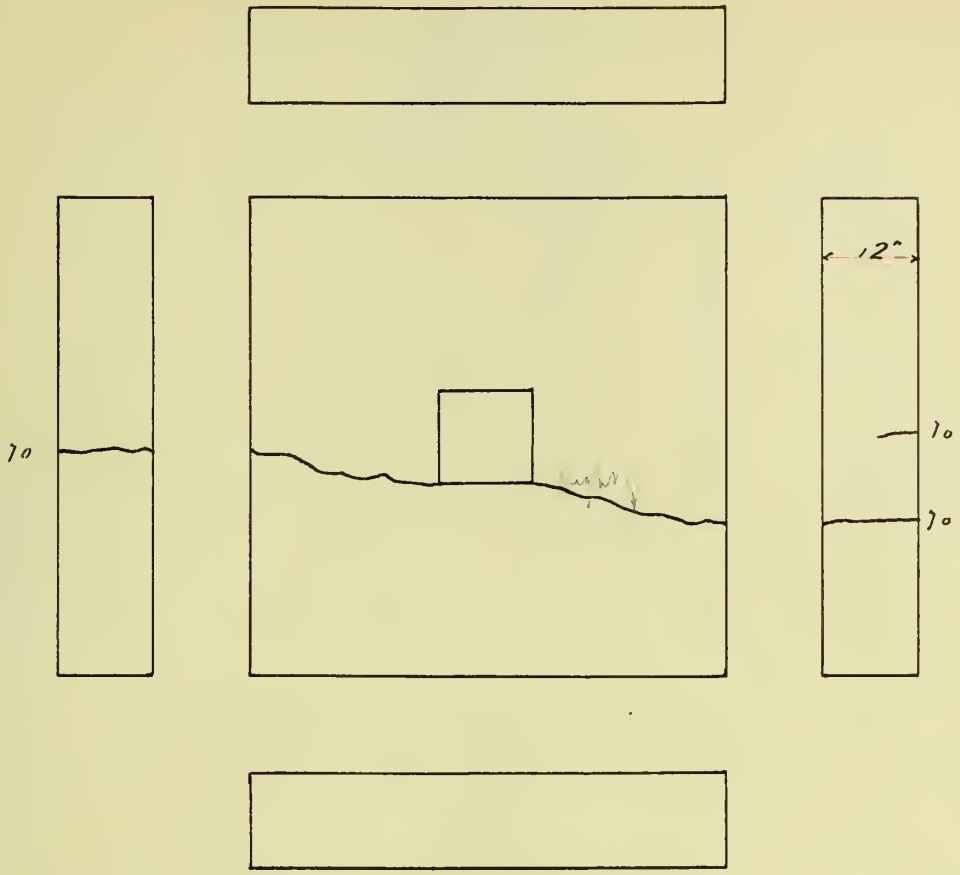
Footing No. 1535 --- 0.6% each way 10 in. deep.

First cracks at 90 T. on all faces opposite edges of pier.

Very gradual failure at 108 T.

Failure was accompanied by the opening of cracks near the corners and the closing of the hair cracks that had appeared near the center of the faces earlier in the test.

This is apparently a failure in the bond between the steel and concrete.



Footing No. 1536 --- 0.6% each way 10 in. deep.

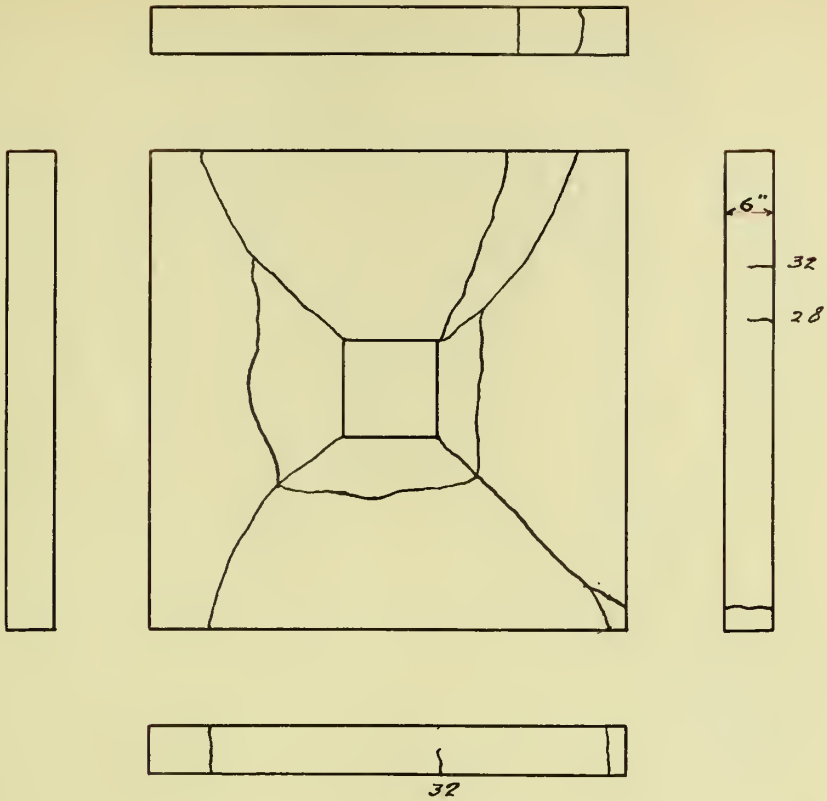
First crack at 70 T.

Very gradual failure at 102 T. Instruments were removed at 90 T.

All indications of failure by slipping of steel.

The cracks across the top were hardly discernible at failure.

Footing was not loaded to total destruction so that it could be taken from the machine intact.



Footing No. 1541 --- 1.2% each way 5 in. deep.

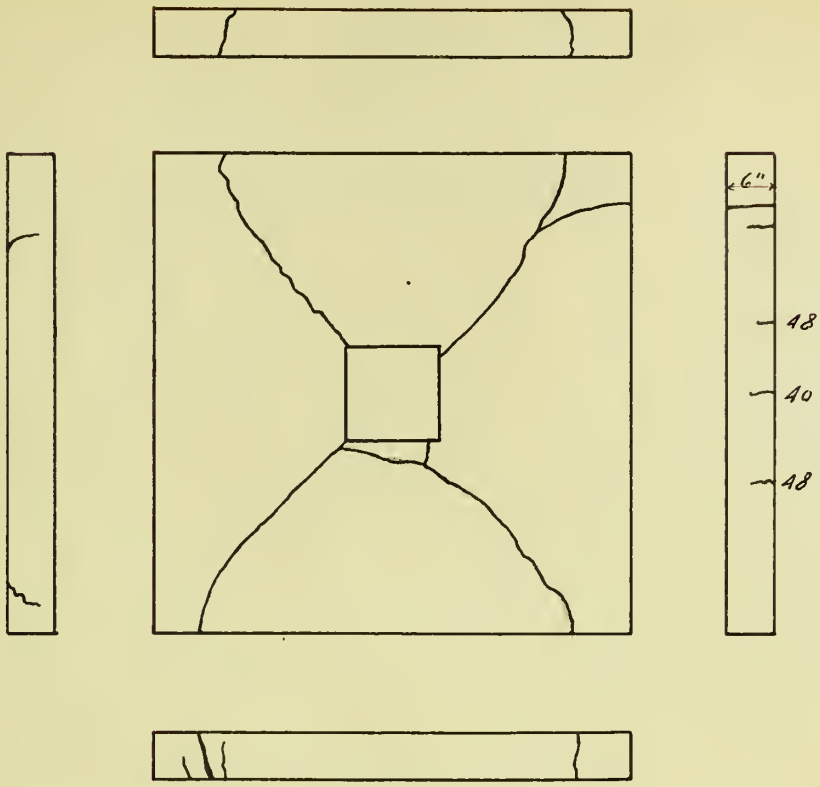
First crack at 28 T.

Instruments were removed at 32 T. The load fell off slightly while instruments were being taken off and could not again be raised above 32T.

The Maximum load 32 T. was held under steady pumping for about two minutes while footing deflected visibly.

There was no severe cracking or punching out accompanied this failure which was perhaps the most gradual of the series.

When load was released some of the concrete below the reinforcing rods dropped off showing that the rods in places had slipped 1/2 to 3/4 of an inch.



Footing No. 1542 --- 1.2% each way 5 in. deep.

First crack at 40 T.

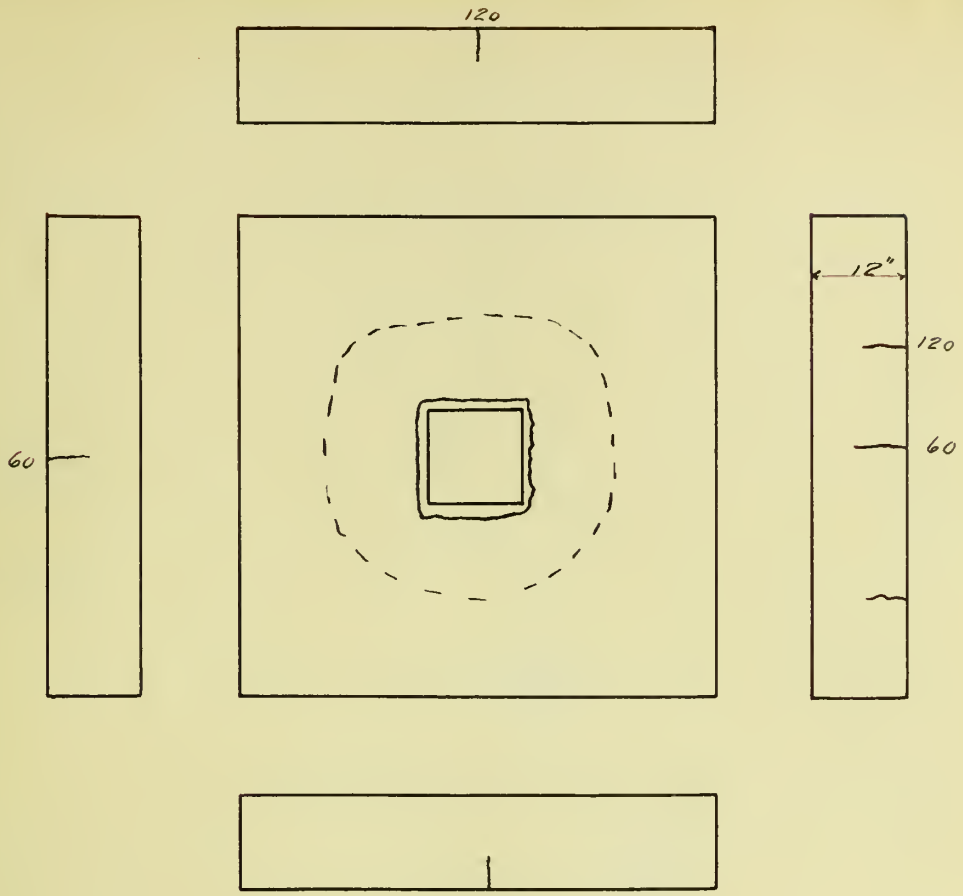
Instruments removed at 40 T.

Gradual failure at 56 T.

This test piece did not hold the maximum load as long as its mate, 1541, but punched in slightly and cracked as shown.

The cracks near the corners on the faces did not open until near the ultimate.

This would show a failure in the bond between the steel and concrete.



Footing No. 1551 --- 0.4% corr. rounds each way, 10 in. deep.

At 60 T. first hair cracks appeared on E. and W. faces.

At 70 T. the E. crack opened very little.

At 80 T. small crack appeared on the S.

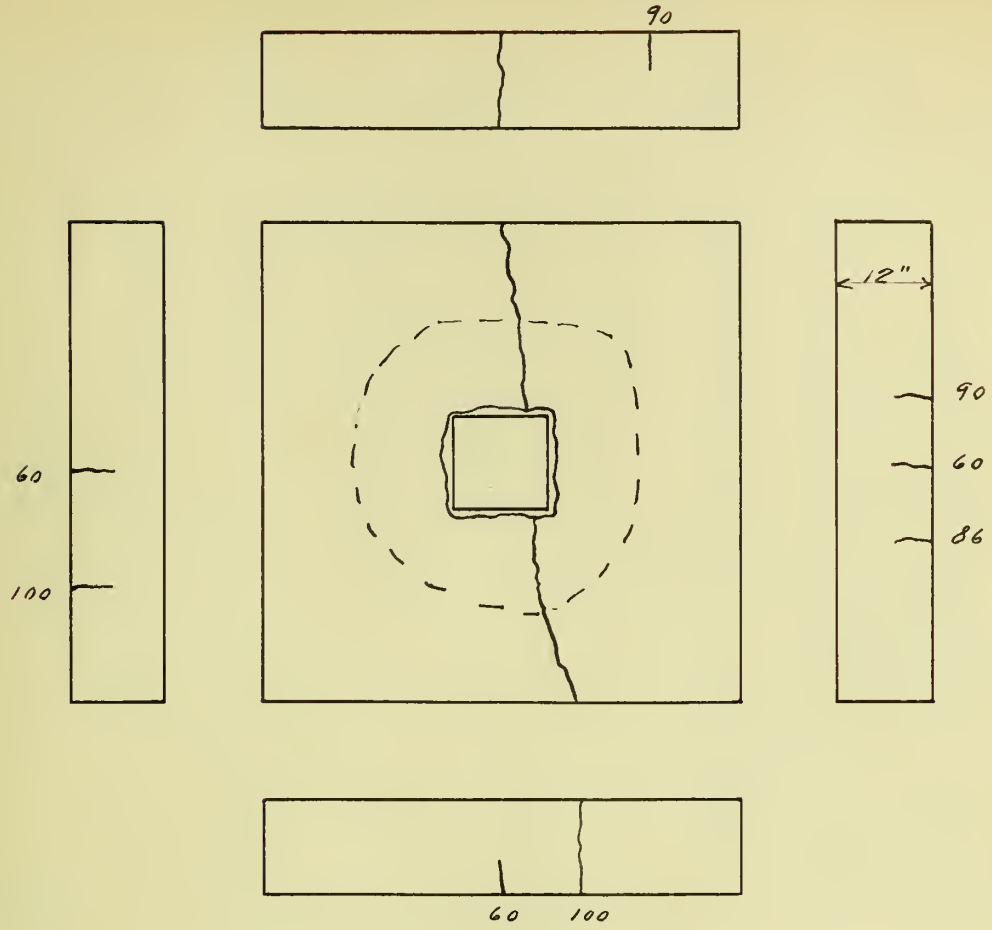
At 90 T. the S. crack plainer.

At 110T. the deflectometers were removed.

At 120 T. small crack appeared in middle of N. face.

At 123 T. the footing failed suddenly by shearing through of the pier while the load was steadily advancing. All the concrete below the steel dropped off.

See photograph on page 94.



Footing No. 1552 --- 0.4% each way 10 in. deep.

First cracks at 60 T.

Instruments removed at 100 T.

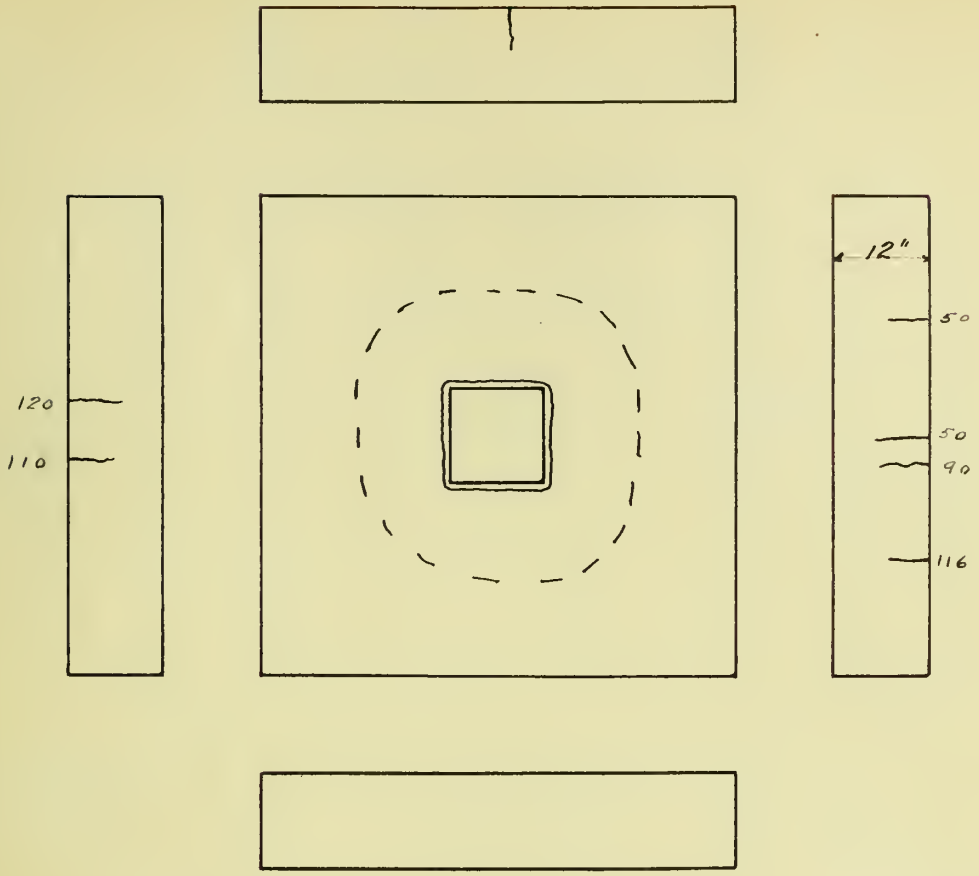
At 110 T. load was released because springs leaned dangerously to E.

Footing was removed from machine and remainder of springs set in.

Footing again mounted and loaded to failure without trouble.

Failure at 128 T. was violent similar to 1551, its mate, except that footing broke in two pieces by north and south crack as shown. (Probably [illegible] concrete)

The shearing out of the pier was accompanied by the dropping off of the concrete below the rods.



Footing No. 1553 --- 0.6% corr. rounds each way 10 in. deep.

At 90 T. first crack appeared on E. face opposite S. edge of pier.

At 100 T. two more cracks appeared on the E. face.

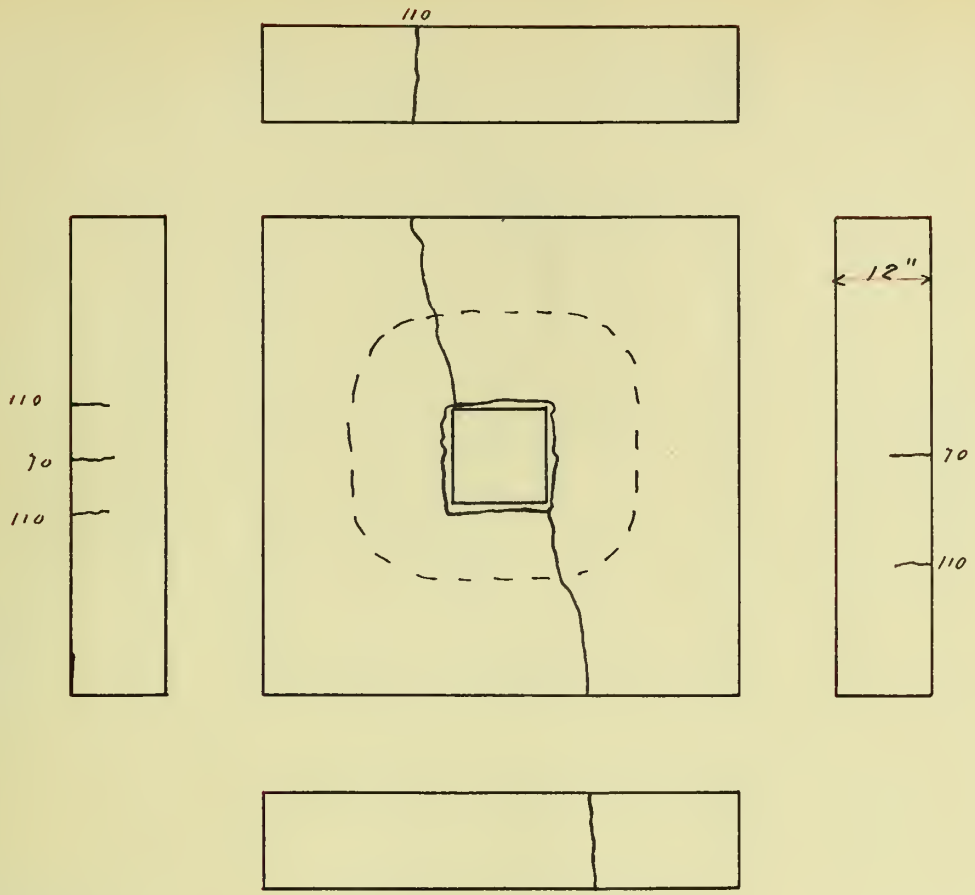
At 110 T, a crack on the N. about the middle and one on the W. near the S. edge of the pier.

At 116 T. another crack on the E. south of the pier.

At 120 T. another crack on the W. near N. edge of pier. The springs were by this time closed to all appearances, the few hair cracks above described being the only indications of failure. The footing was therefore put aside until the remainder of the springs could be inserted.

The footing was again loaded four weeks later with the remainder of the springs in place.

At 174 T. the pier sheared through suddenly and, except for the preliminary crumbling of the pier that took place in 1531, the failure was almost like that footing.

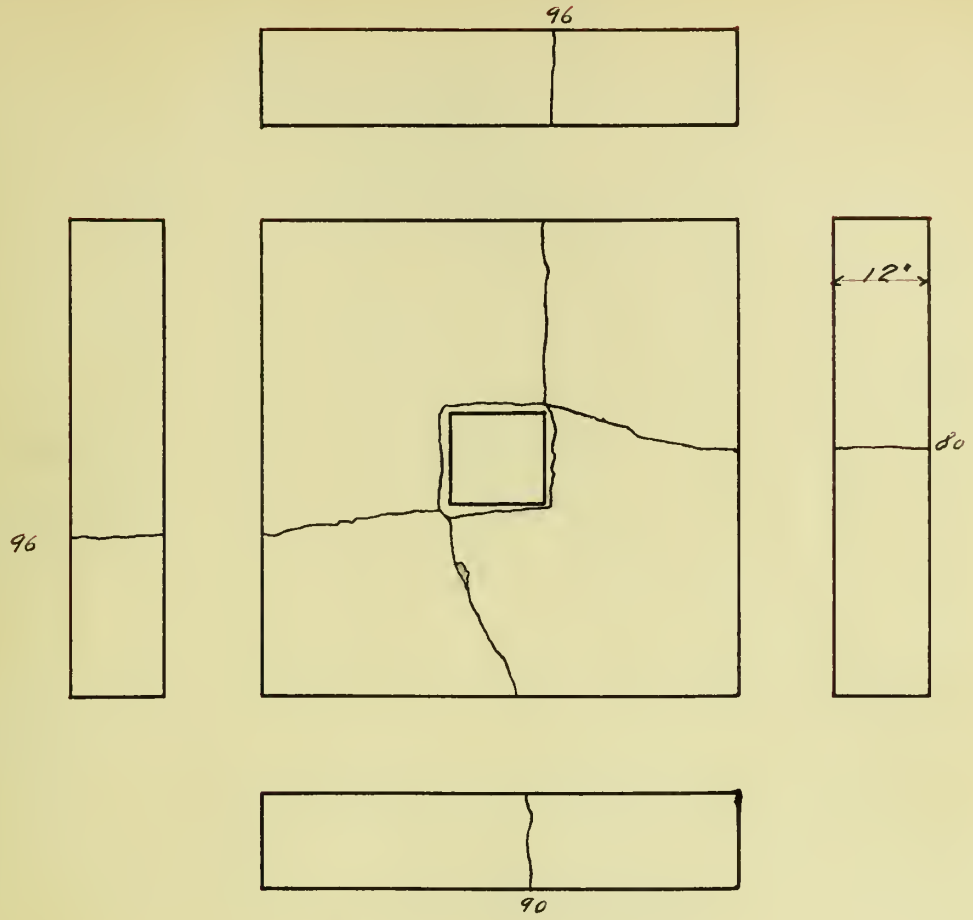


Footing No. 1554 --- 0.6% corr.squares each way 10 in. deep.

First crack at 70 T.

Final failure was by shearing through of the pier as in its mate 1553. Besides this a crack appeared across the top of the footing from N. to S.

The angle of shear was about 45 degrees from the base of the pier outwards.



Footing No. 1561 --- Same amount of steel as in 1531 spaced uniformly in four directions 10 in. deep.

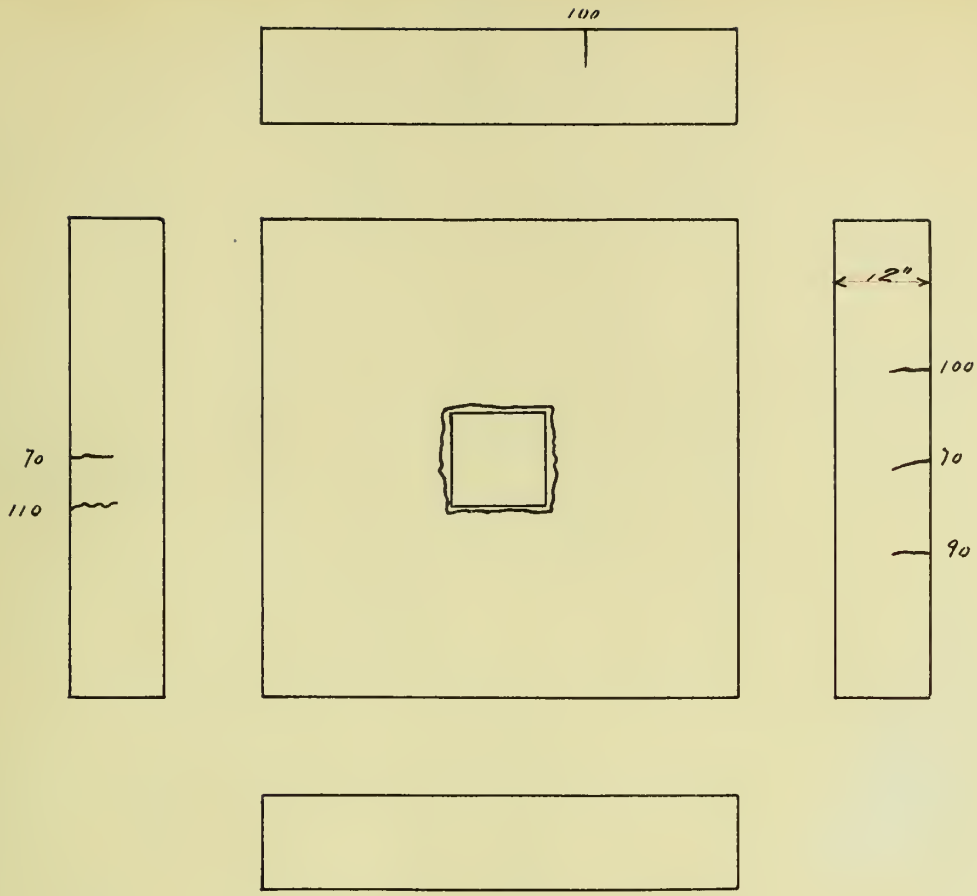
First crack appeared at 80 T. and others as indicated on diagram.

At 106 T. the upper beam tilted dangerously and the springs began to lean to the N. Load was released and the centering changed slightly.

Footing was again loaded with same result so the test was discontinued to readjust the apparatus. The trouble was found to be in the lower I-beams of the foundation which were not plumb under the jacks.

The next day the footing was again loaded without instruments. Failure occurred gradually, the first indications being the opening of the cracks on all faces. This was followed by a gradual shearing through of the pier.

The load was released before the footing was completely demolished.



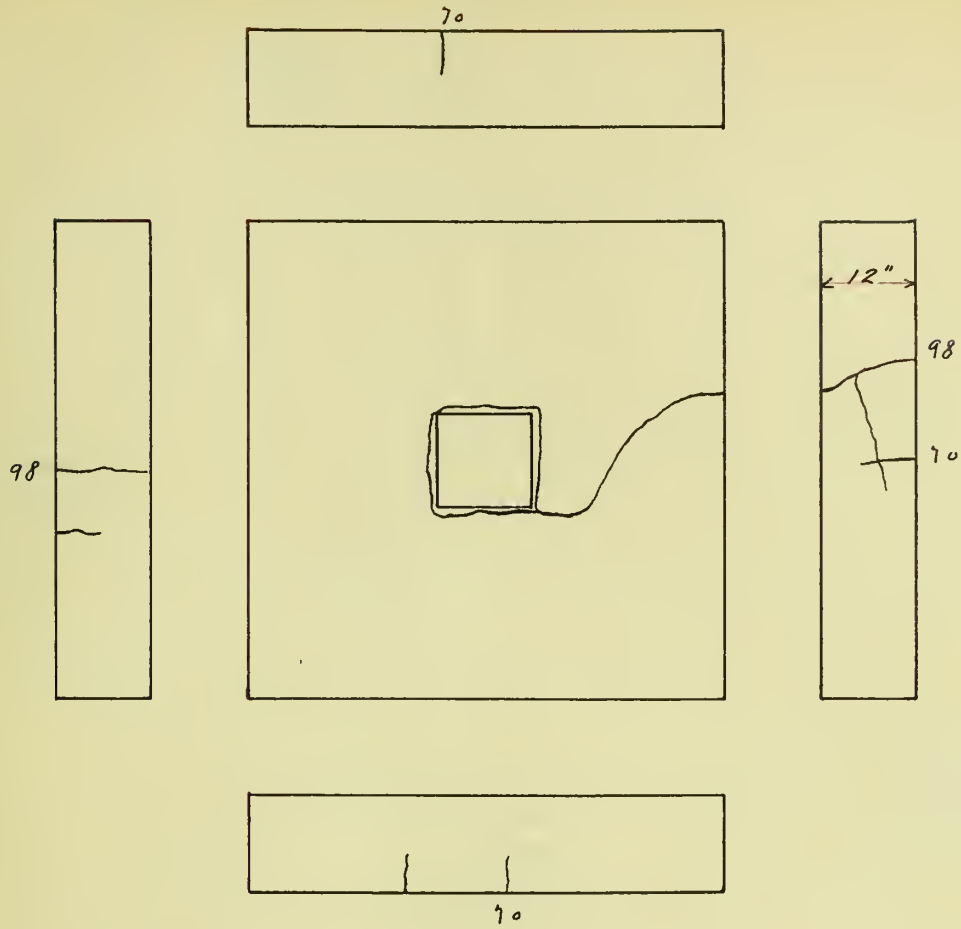
Footing No. 1562 --- Same amount of steel as in 1531 spaced uniformly in four directions. 10 in. deep.

First crack at 70 T.

Insruments removed at 110 T.

Failure at 116 T. by gradual opening of cracks on E. and W. faces followed, after slight falling off of load, by sudden shearing through of column.

No cracks showed on top after failure.



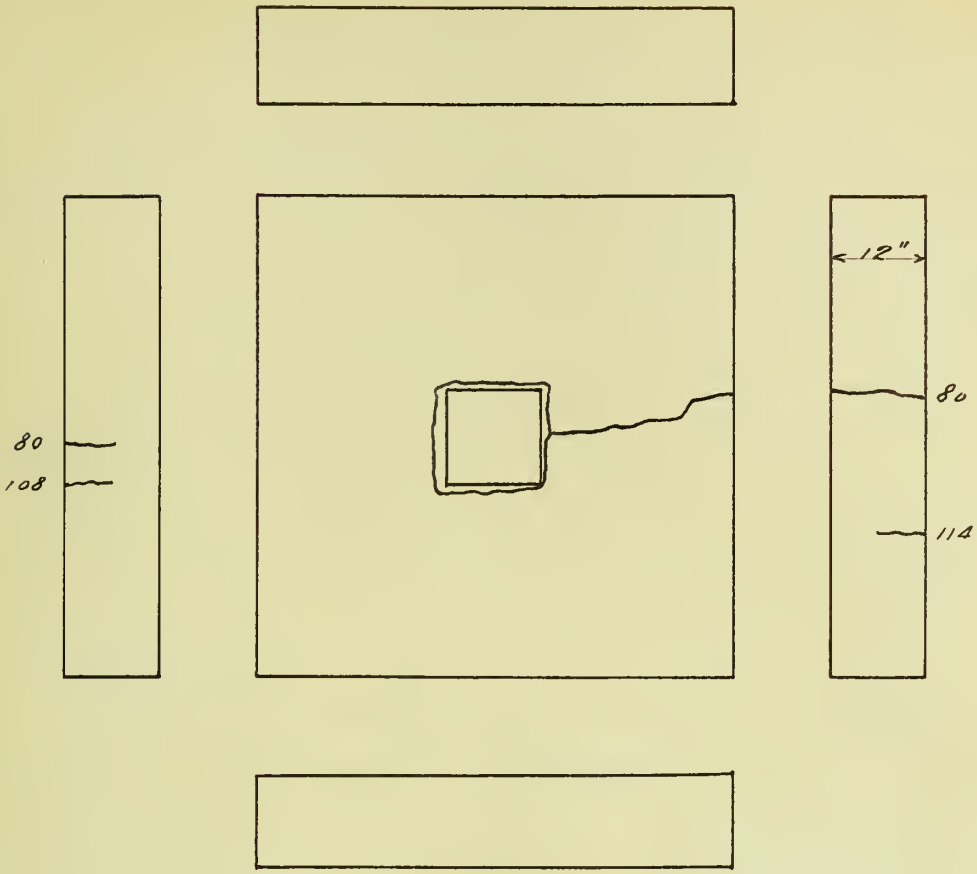
Footing No. 1563 --- Same as 1561 except omit three short diagonal rods in corners. 10 in. deep.

First crack at 70 T. and others as shown on diagram.

Failure occurred at maximum load of 90 T. by gradual opening of cracks on E. and W. faces.

After sustaining the load for about two minutes, the pier sheared through and the crack on the E. extended across the footing to the pier as shown.

A horizontal crack appeared on the E. face at the maximum load but closed again after failure.



Footing No. 1564 --- Same as 1561 except omit three short diagonal rods in corners.

First crack at 80 T.

Instruments removed at 80 T.

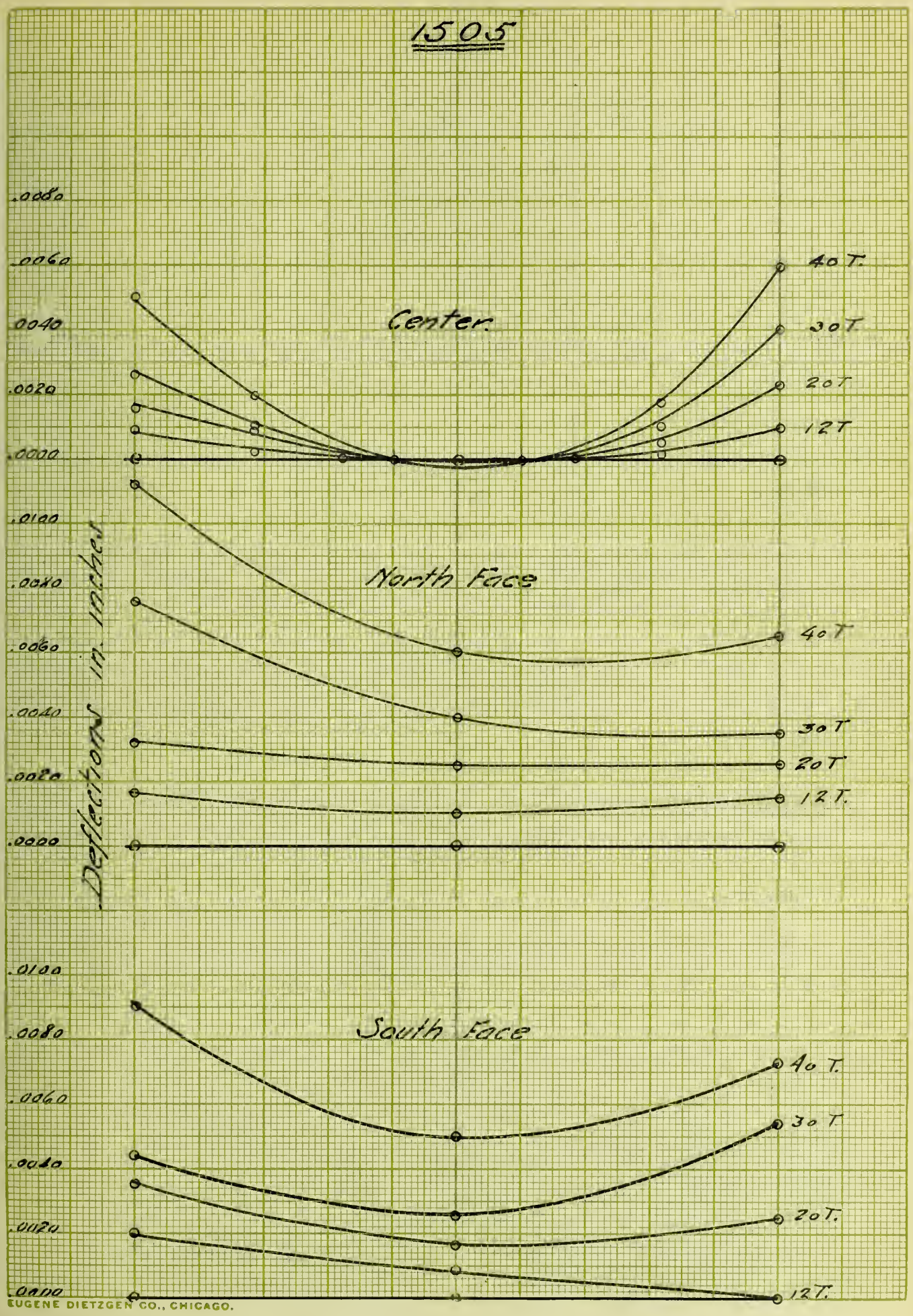
Failure at 116 T. by gradual opening of cracks followed by sudden shearing through of pier.

Explanation of Graphs.

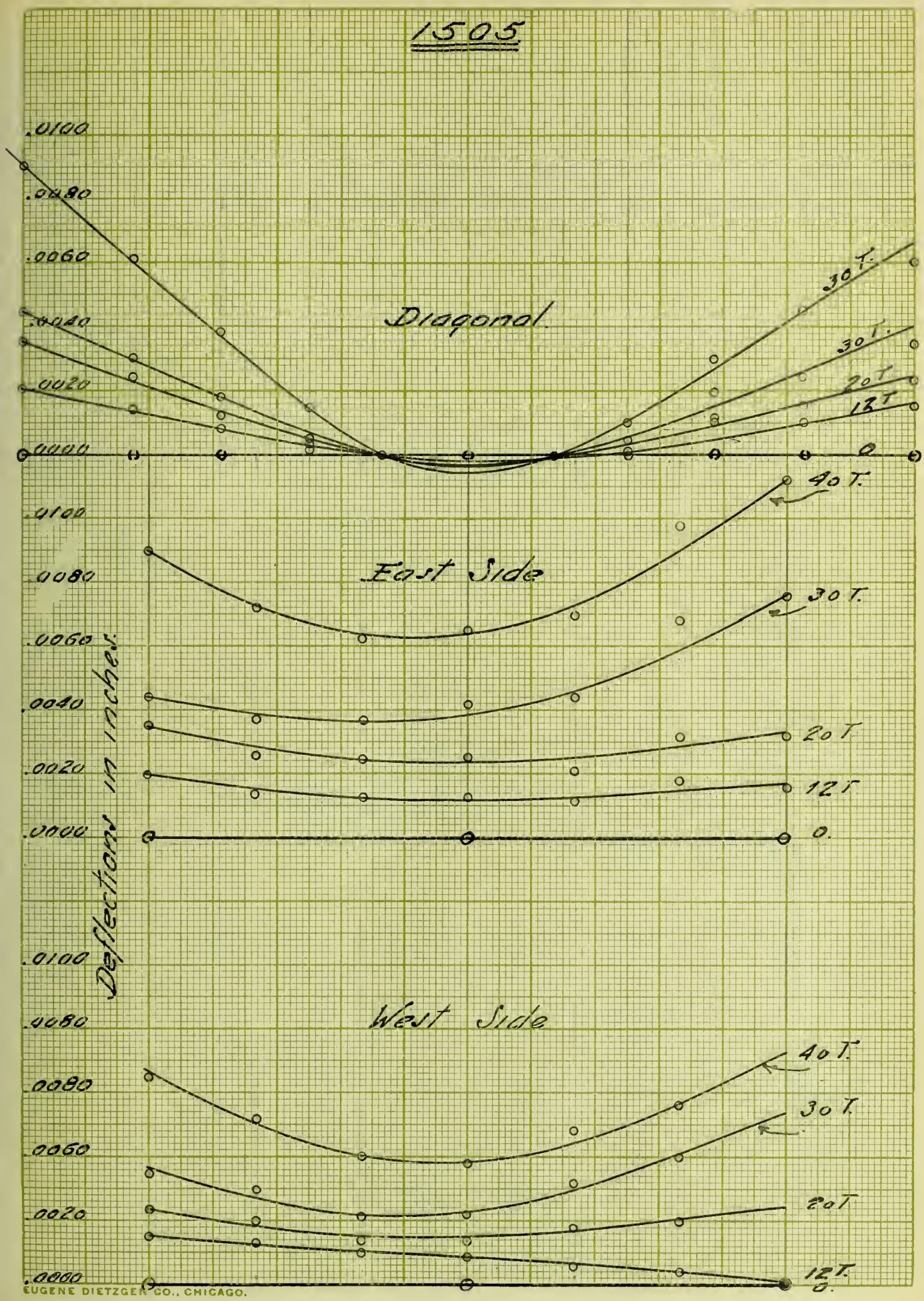
The graphs on pages 48 to 76 show the actual deflections of the various points on the footing with respect to a zero section at the center determined by the three points of support of the frame to which the instruments are attached. On the base line for each set of graphs the spacing of the deflectometers is drawn to scale and the ordinates at those points represent the deflections at a greatly exaggerated scale. A curve is drawn for each increment in the load,except where the curves would be too close to be distinguishable.

The graphs on pages 77 to 83 show the variation of the deflections at certain of the points with the load. Three graphs were plotted for each of the footings, one is the average of the deflectometer readings at the corners, one is the average at the middle of the east and west sides, and the third is the average at the pier.

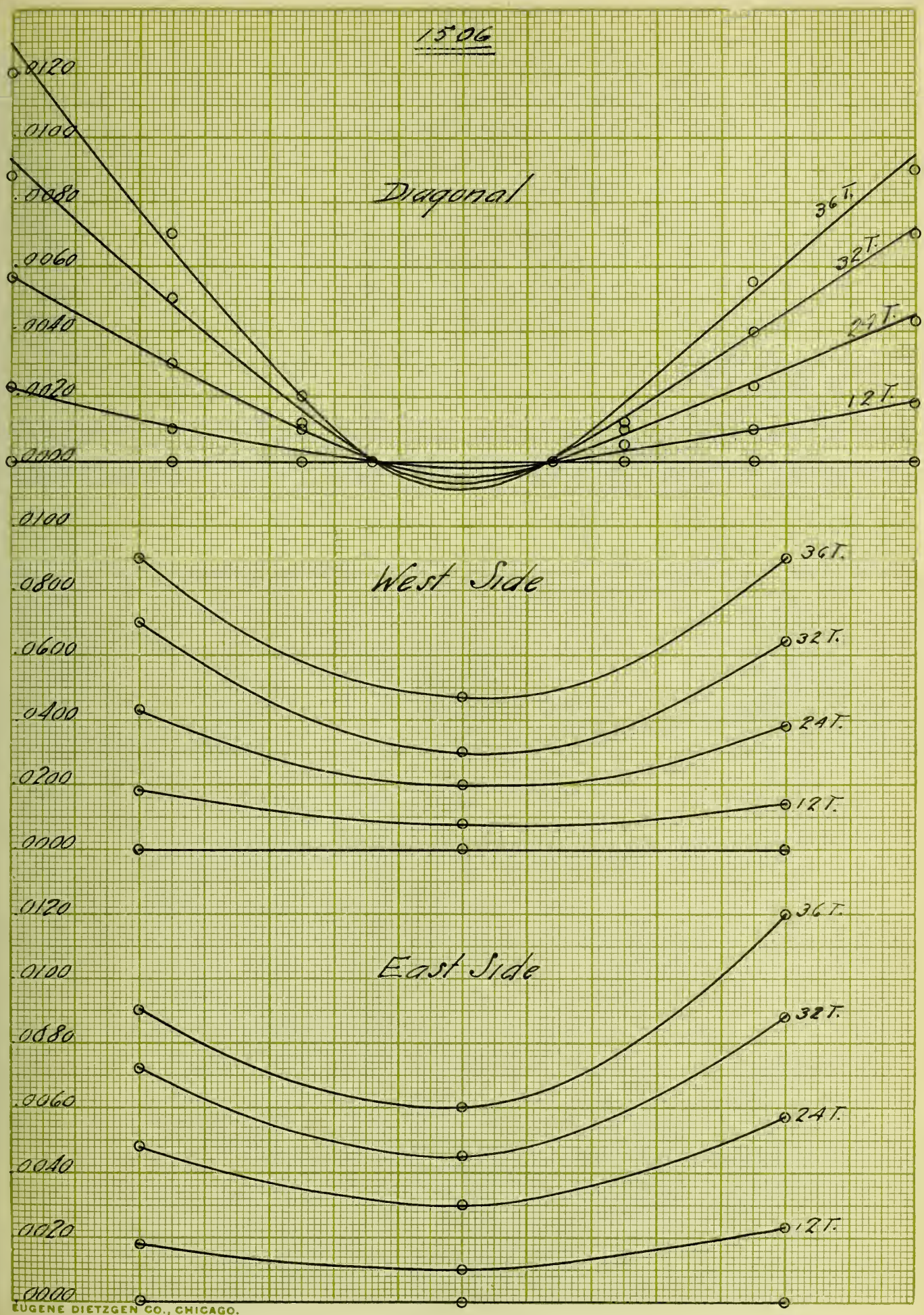
The deflectometers were always set to zero before the load was applied in order to facilitate the reading and recording of the deflections.



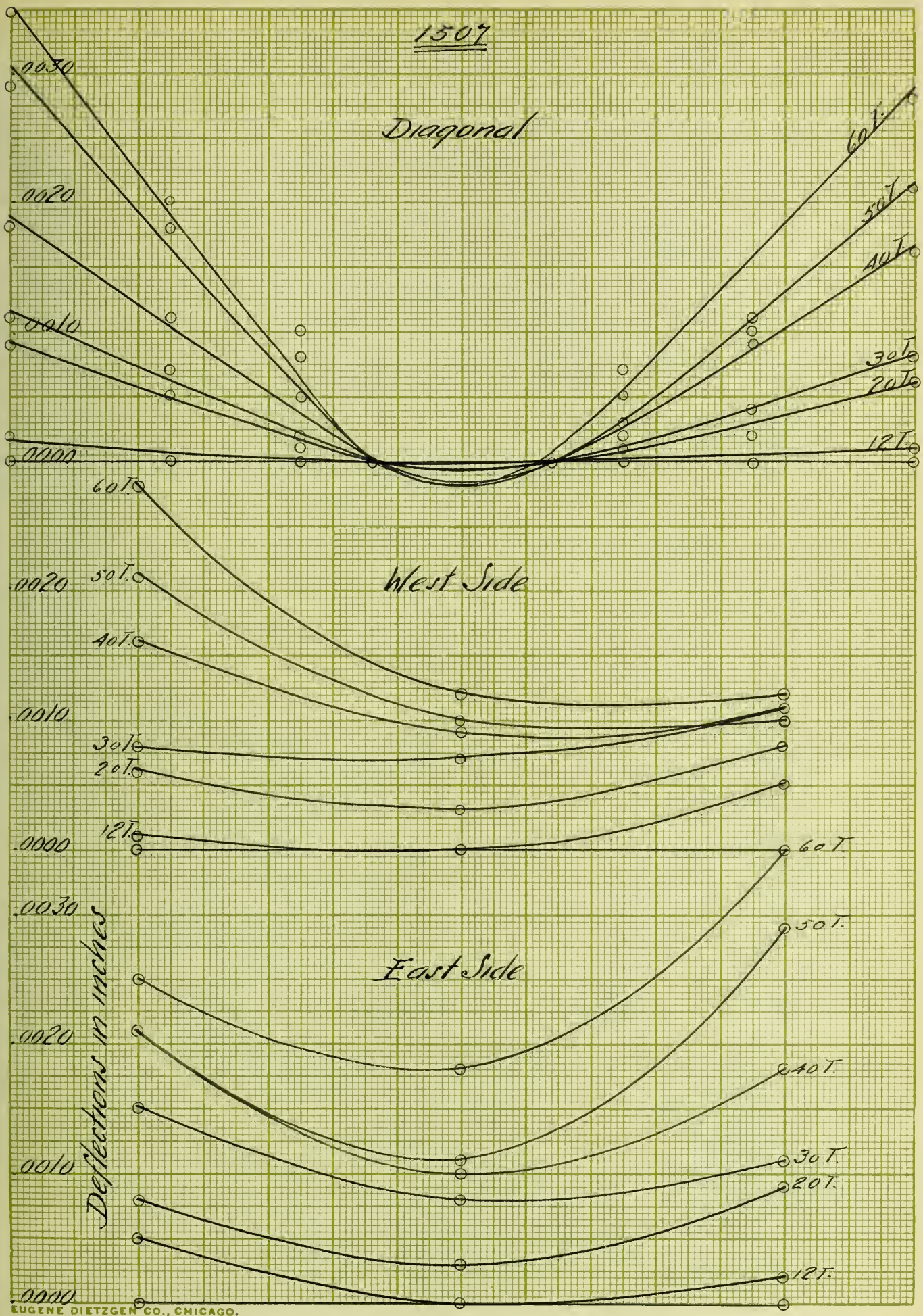
EUGENE DIETZGEN CO., CHICAGO.



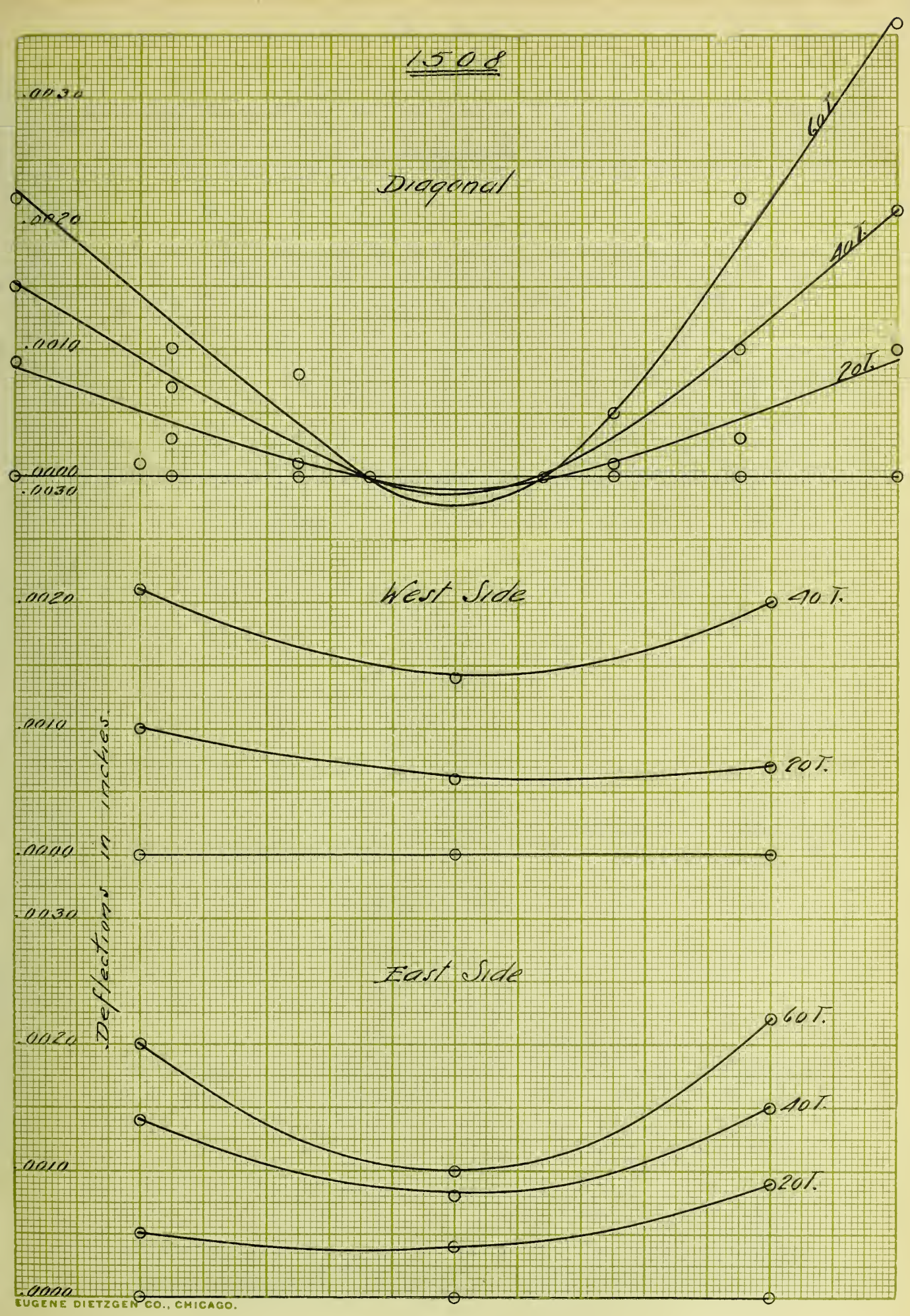
EUGENE DIETZGEN CO., CHICAGO.



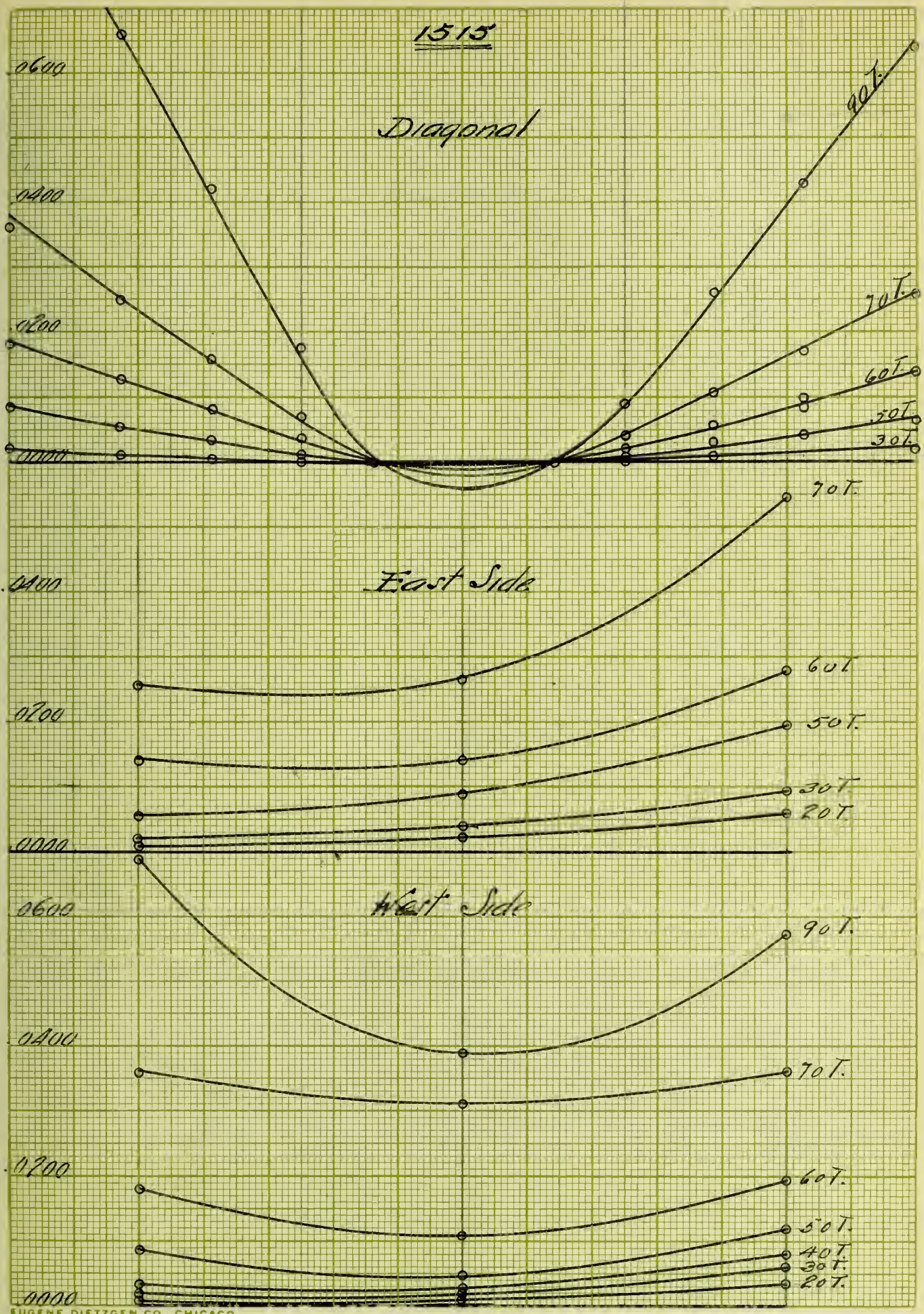
EUGENE DIETZGEN CO., CHICAGO.



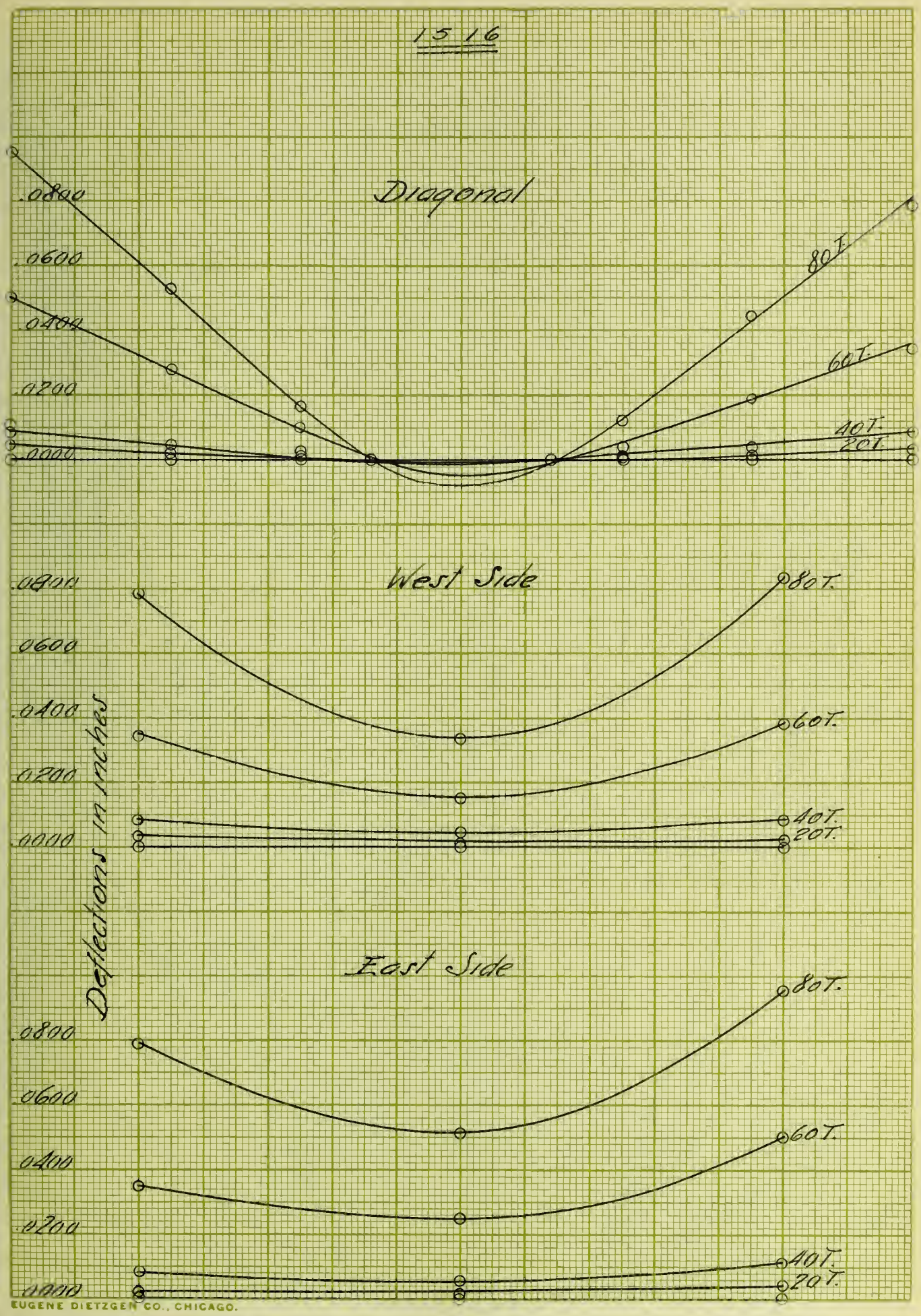
EUGENE DIETZGEN CO., CHICAGO.



EUGENE DIETZGEN CO., CHICAGO.



EUGENE DIETZGEN CO., CHICAGO.


15 16
Diagonal
.0800
.0600
.0400
.0200
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80T.
60T.
40T.
20T.
West Side
.0800
.0600
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Deflections in inches
East Side
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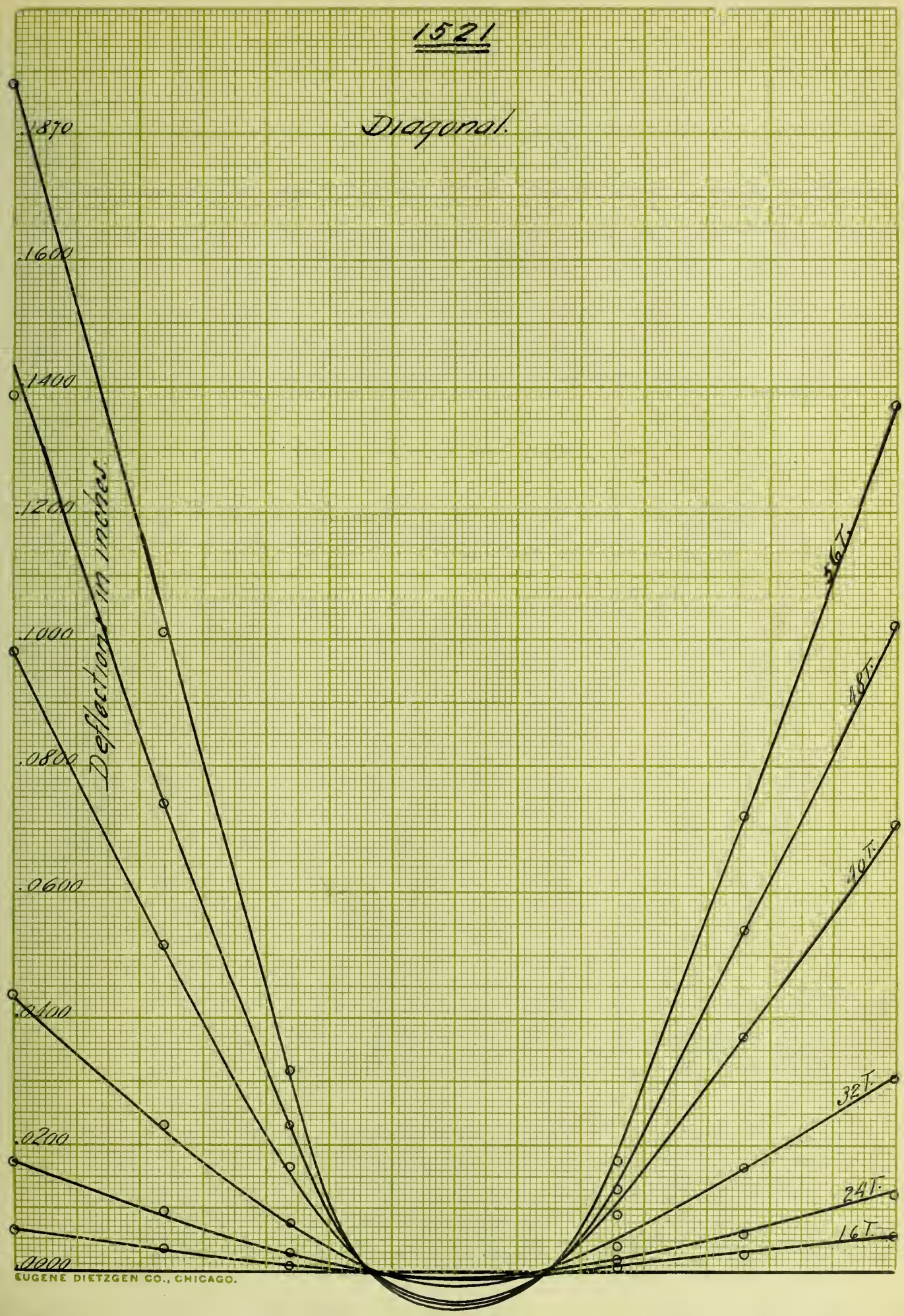
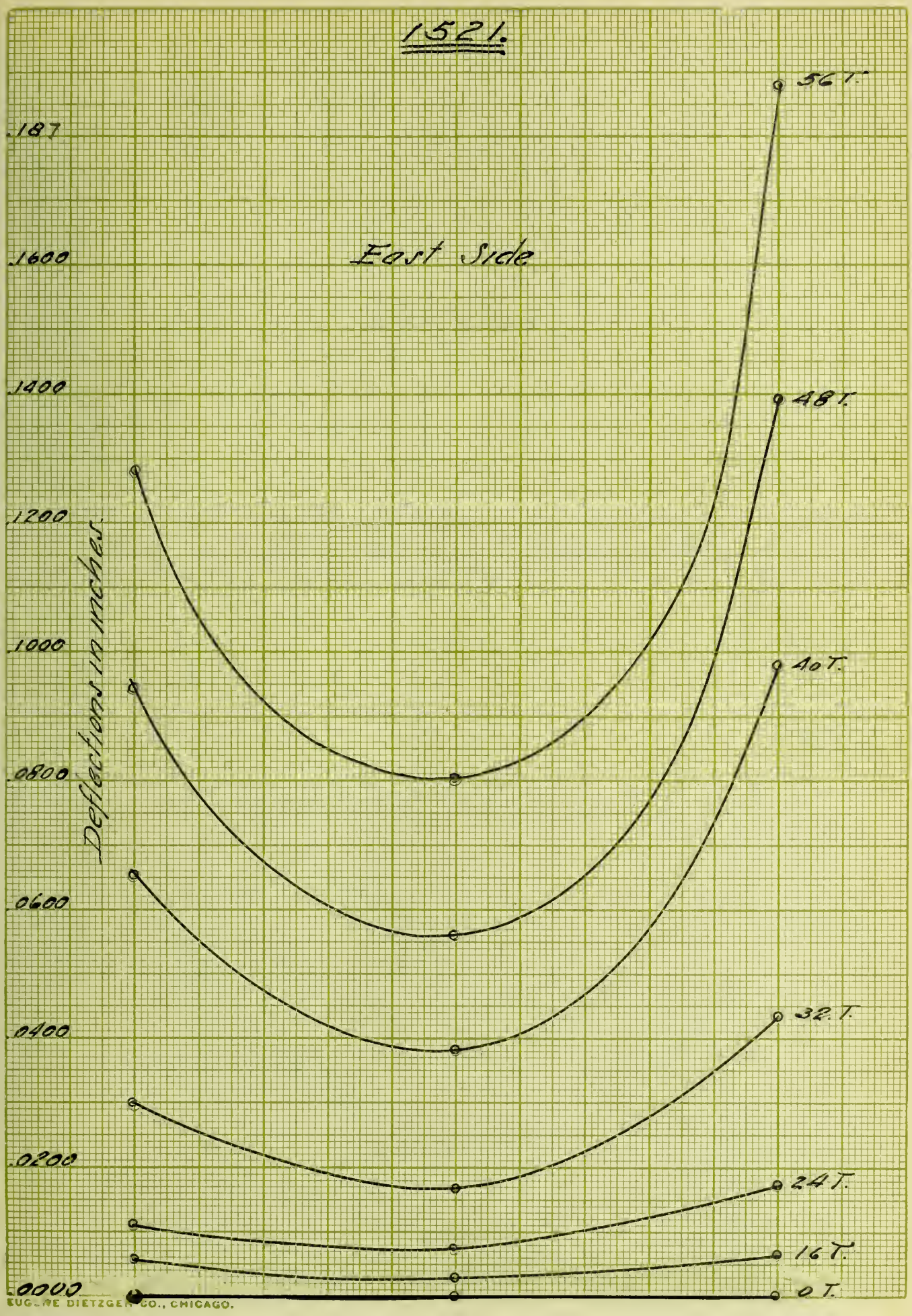
20T.
EUGENE DIETZGEN CO., CHICAGO.


1521
Diagonal.
.1870
.1600
.1400
.1200
.1000
.0800
.0600
.0400
.0200
.0000
Deflection in inches.
56T.
48T.
40T.
32T.
24T.
16T.
EUGENE DIETZGEN CO., CHICAGO.


1521.
East Side
Deflections in inches.
.1870
.1600
.1400
.1200
.1000
.0800
.0600
.0400
.0200
.0000
56 T.
48 T.
40 T.
32 T.
24 T.
16 T.
0 T.
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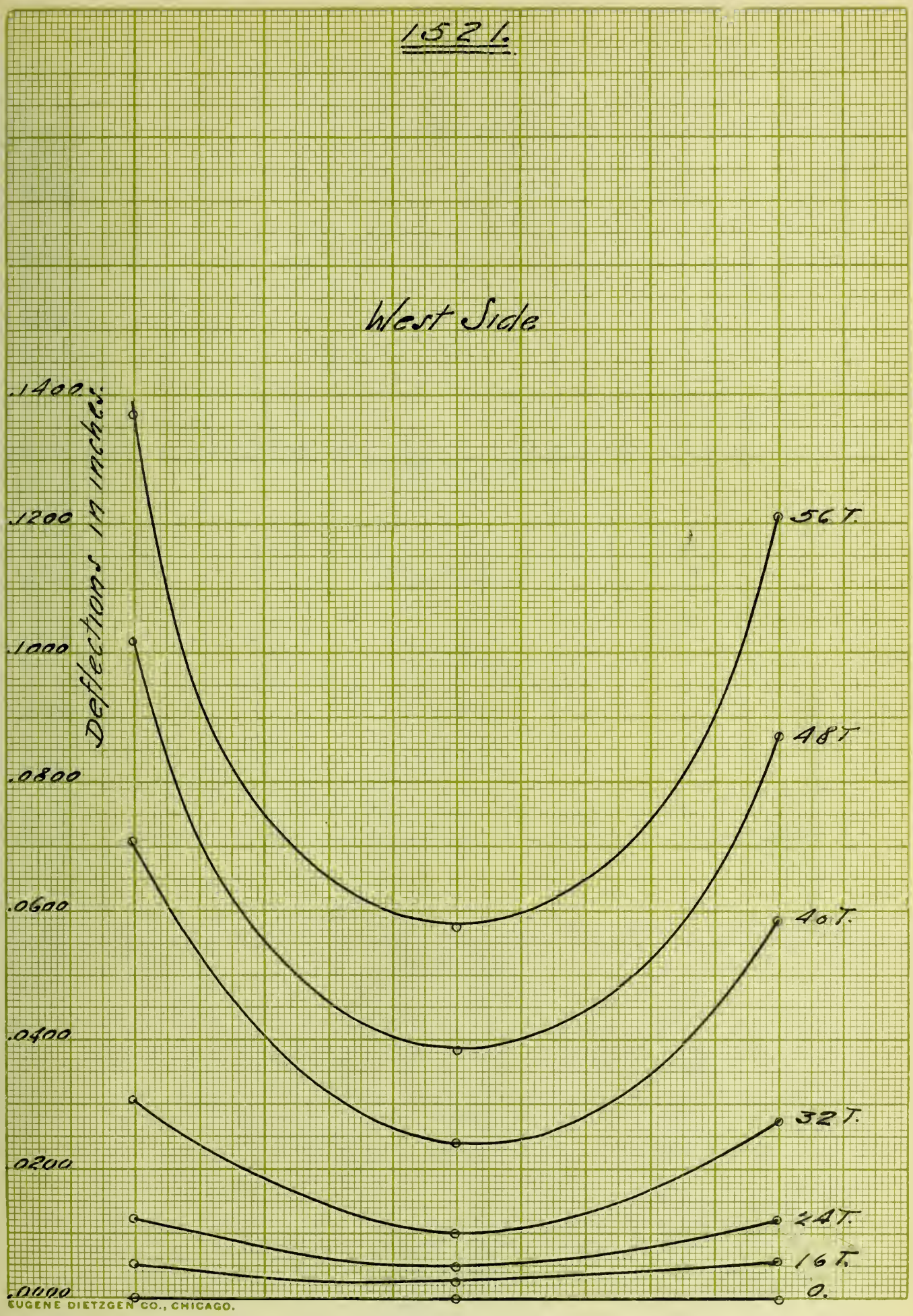
1521.
West Side
Deflections in inches.
.1400
.1200
.1000
.0800
.0600
.0400
.0200
.0000
56 T.
48 T.
40 T.
32 T.
24 T.
16 T.
0.
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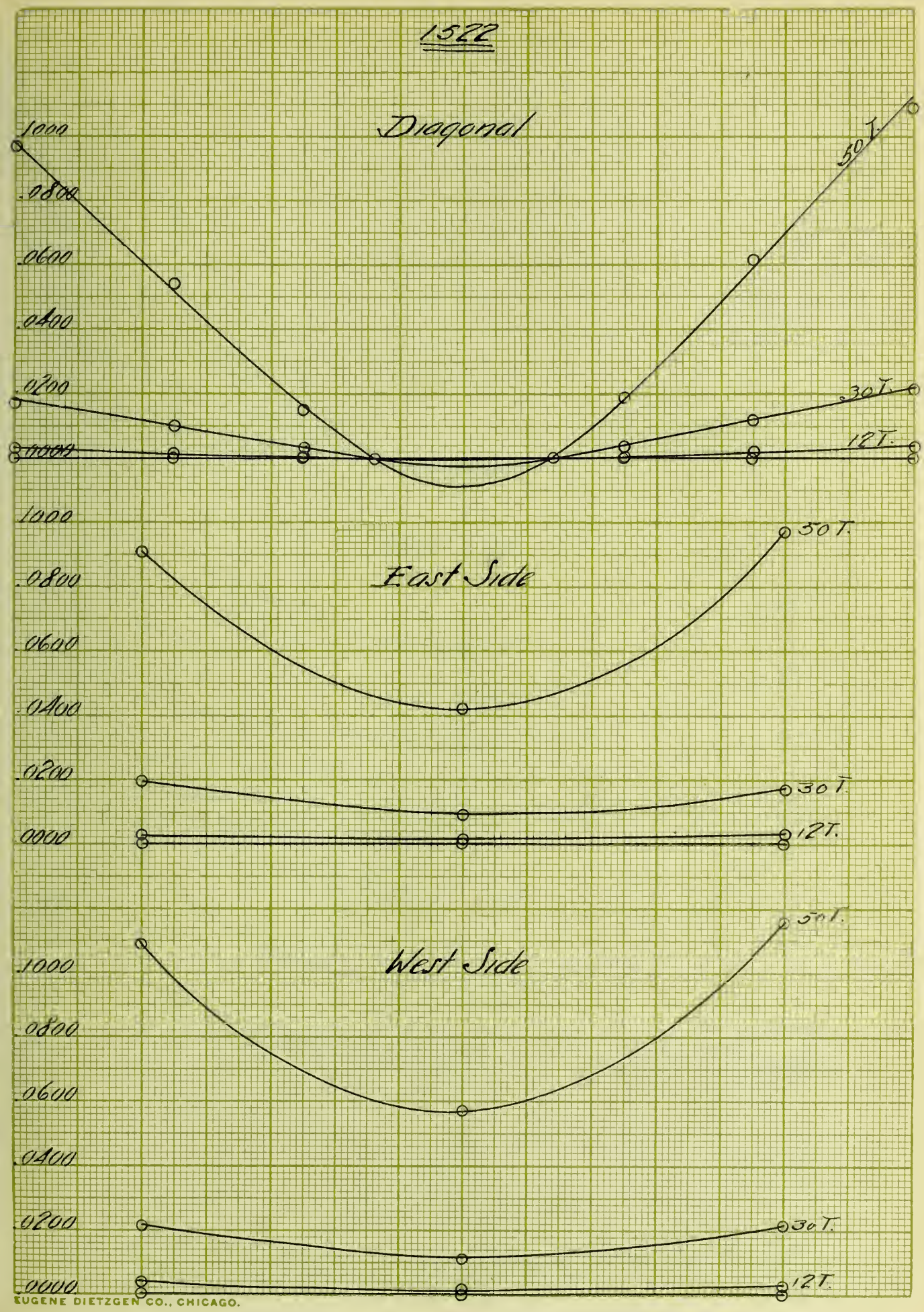

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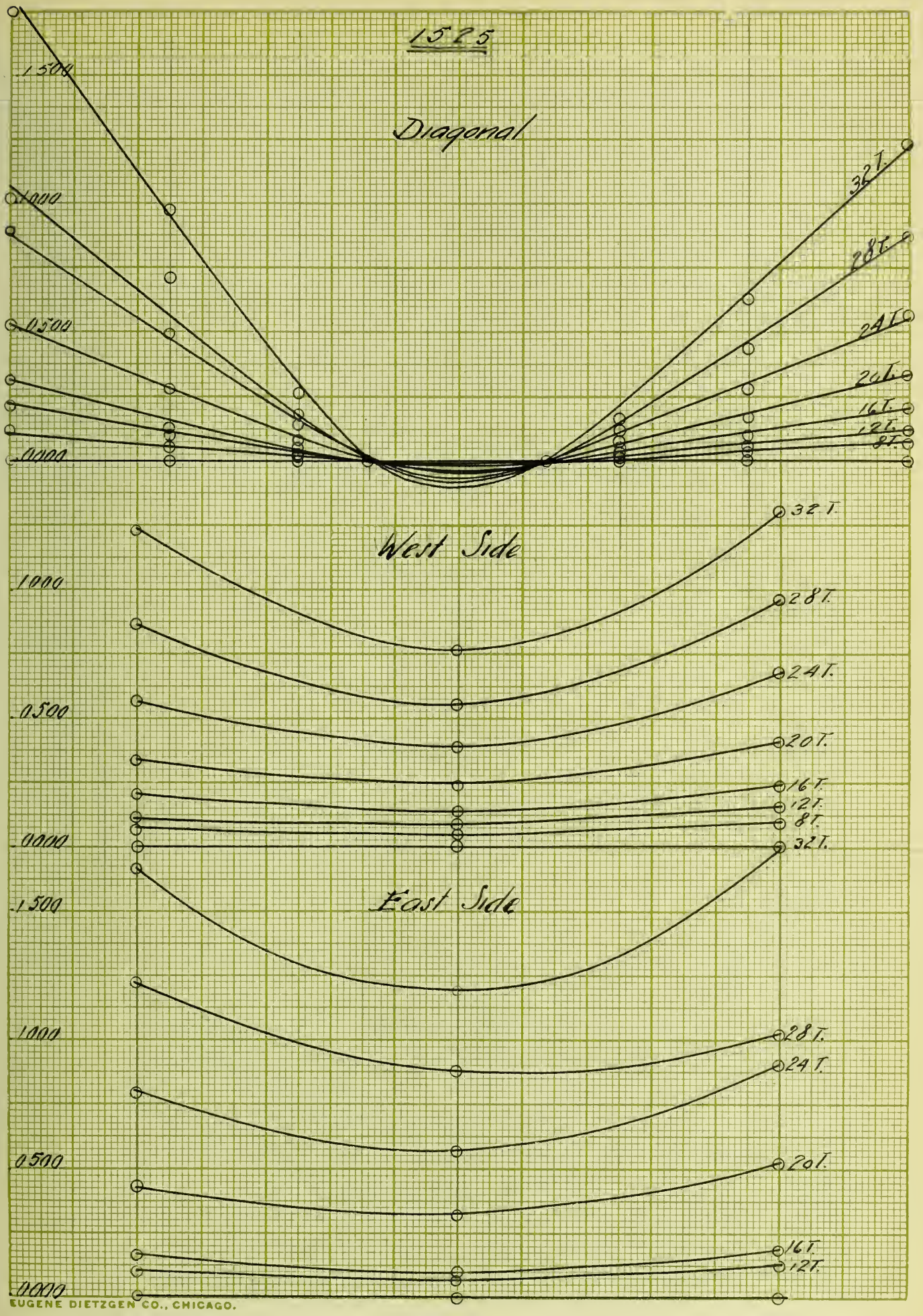

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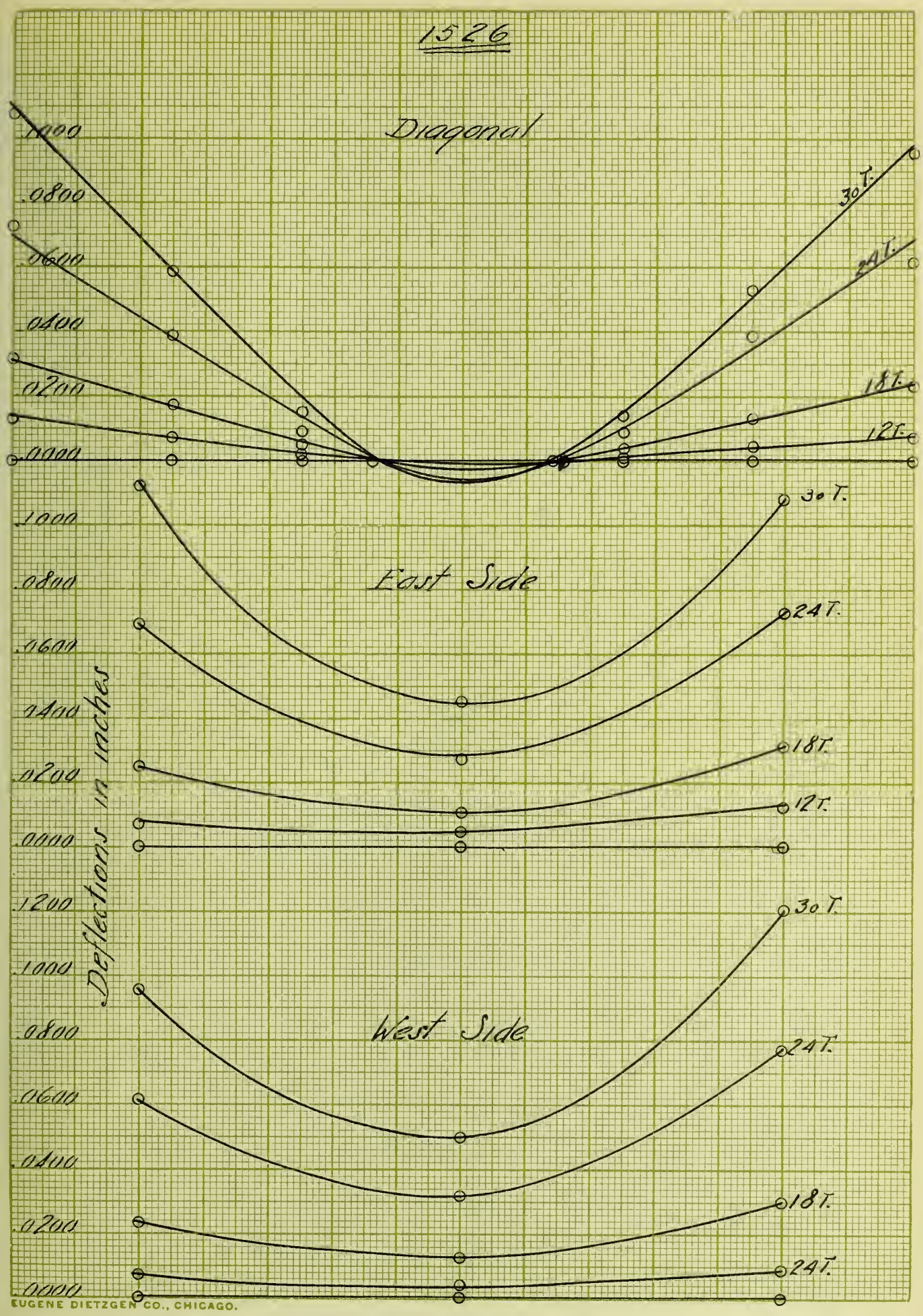

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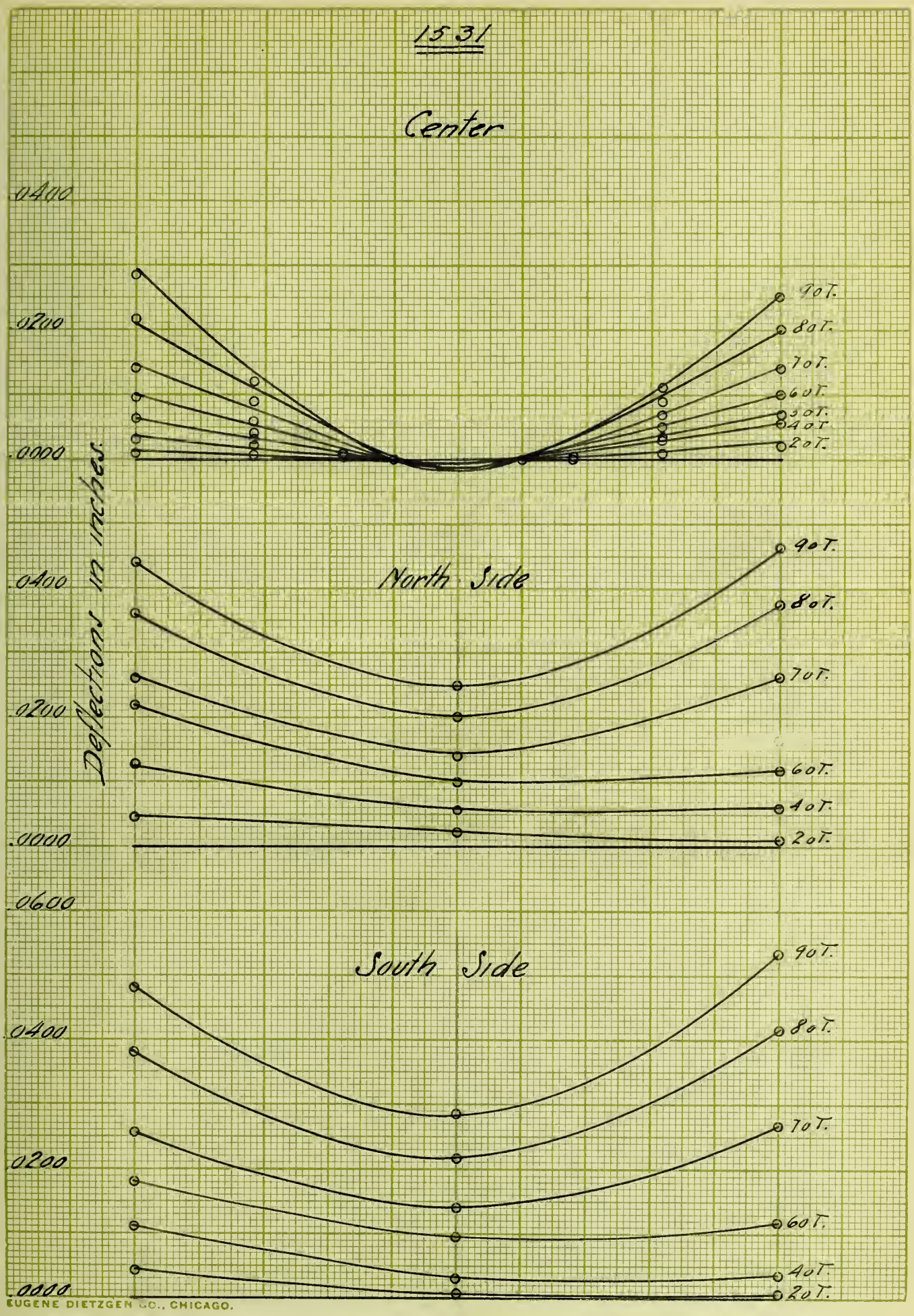

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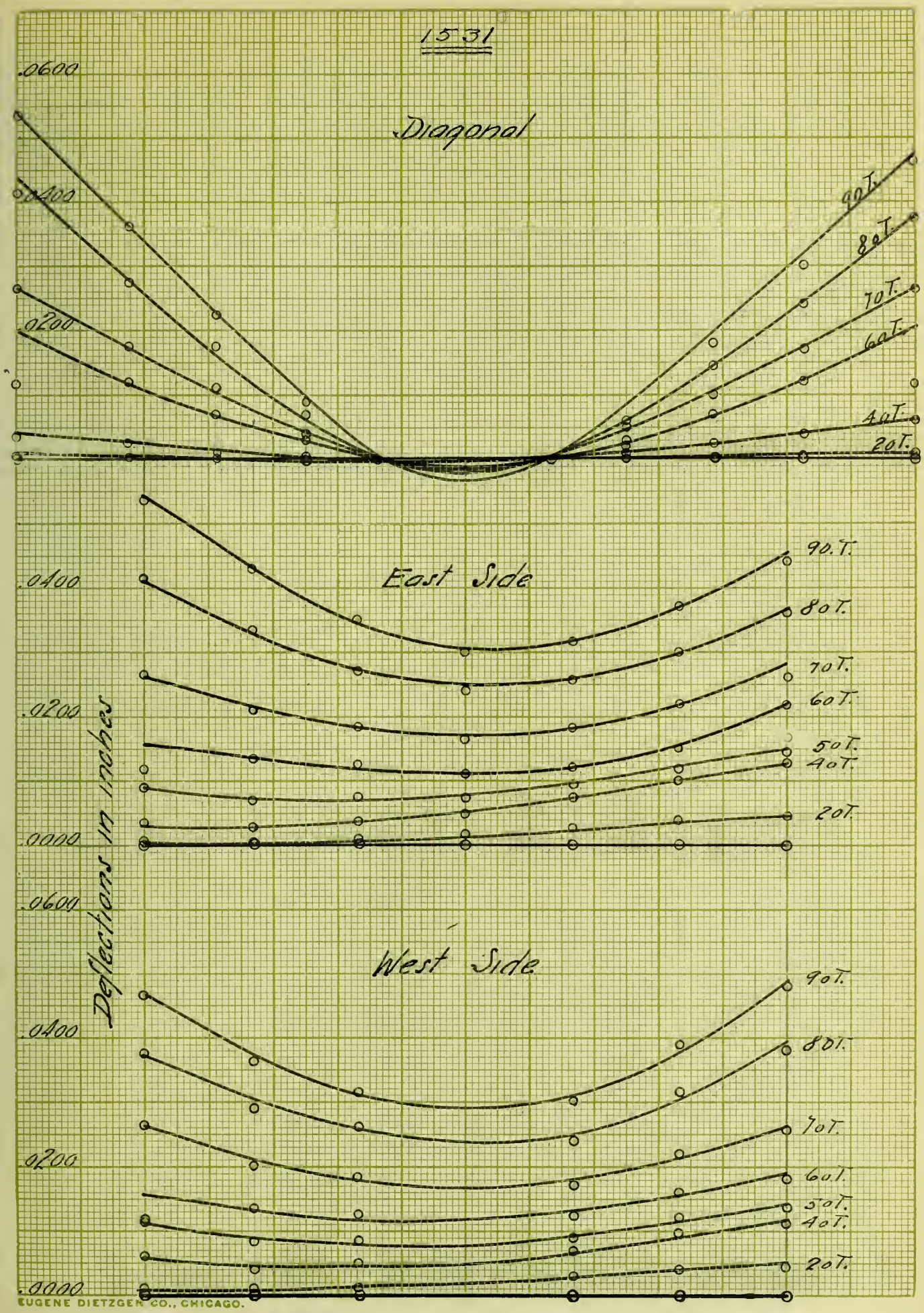

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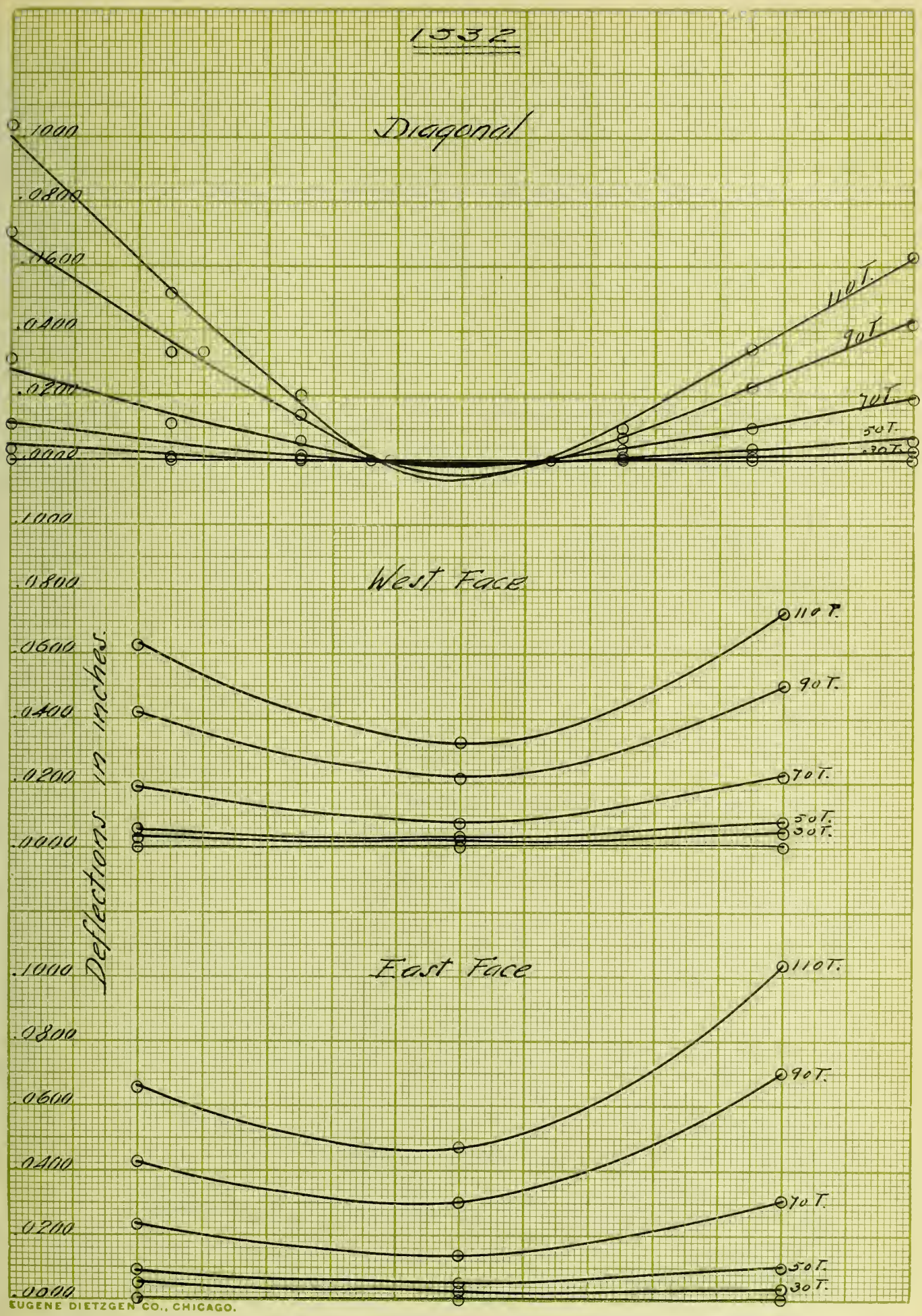

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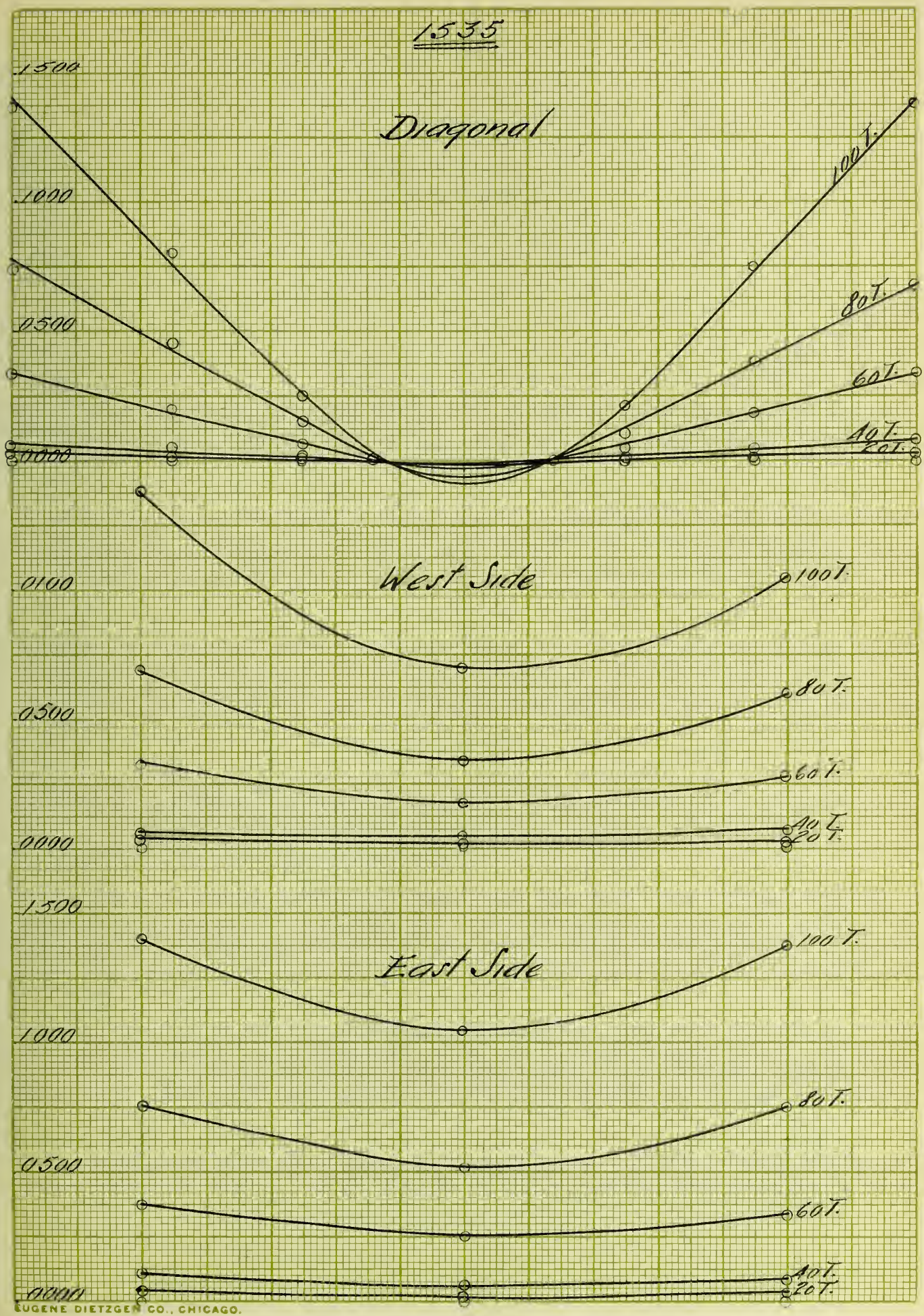

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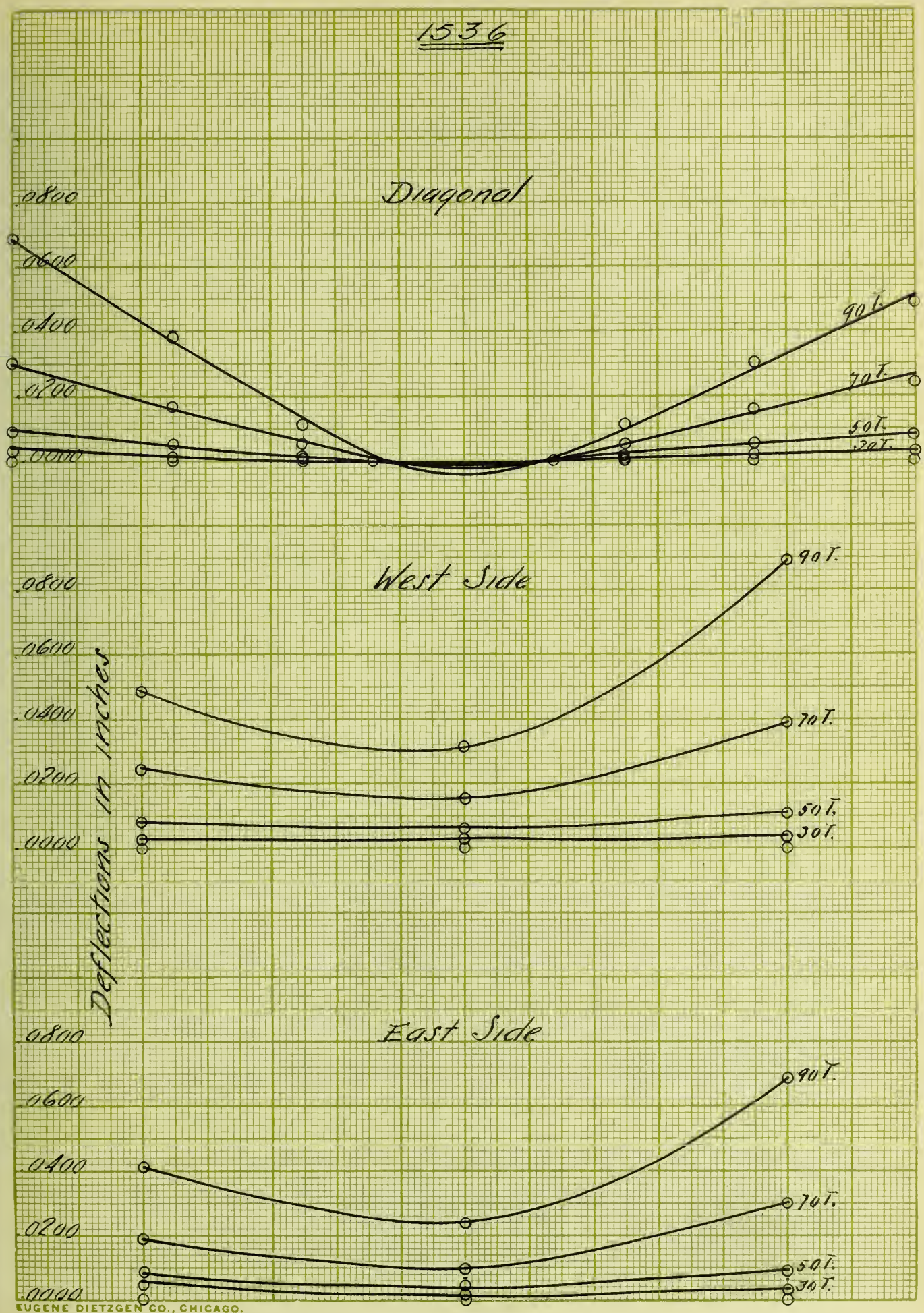

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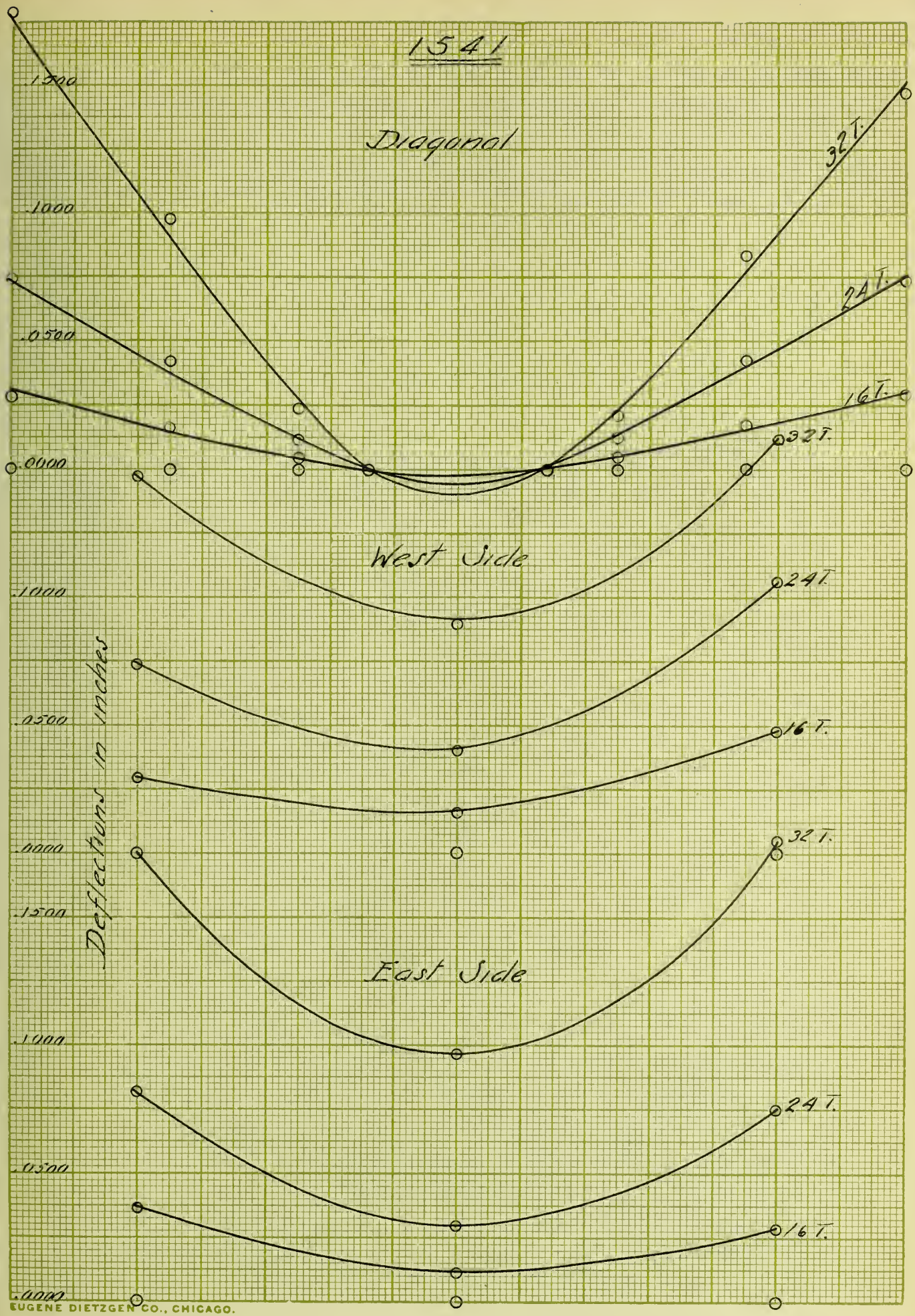

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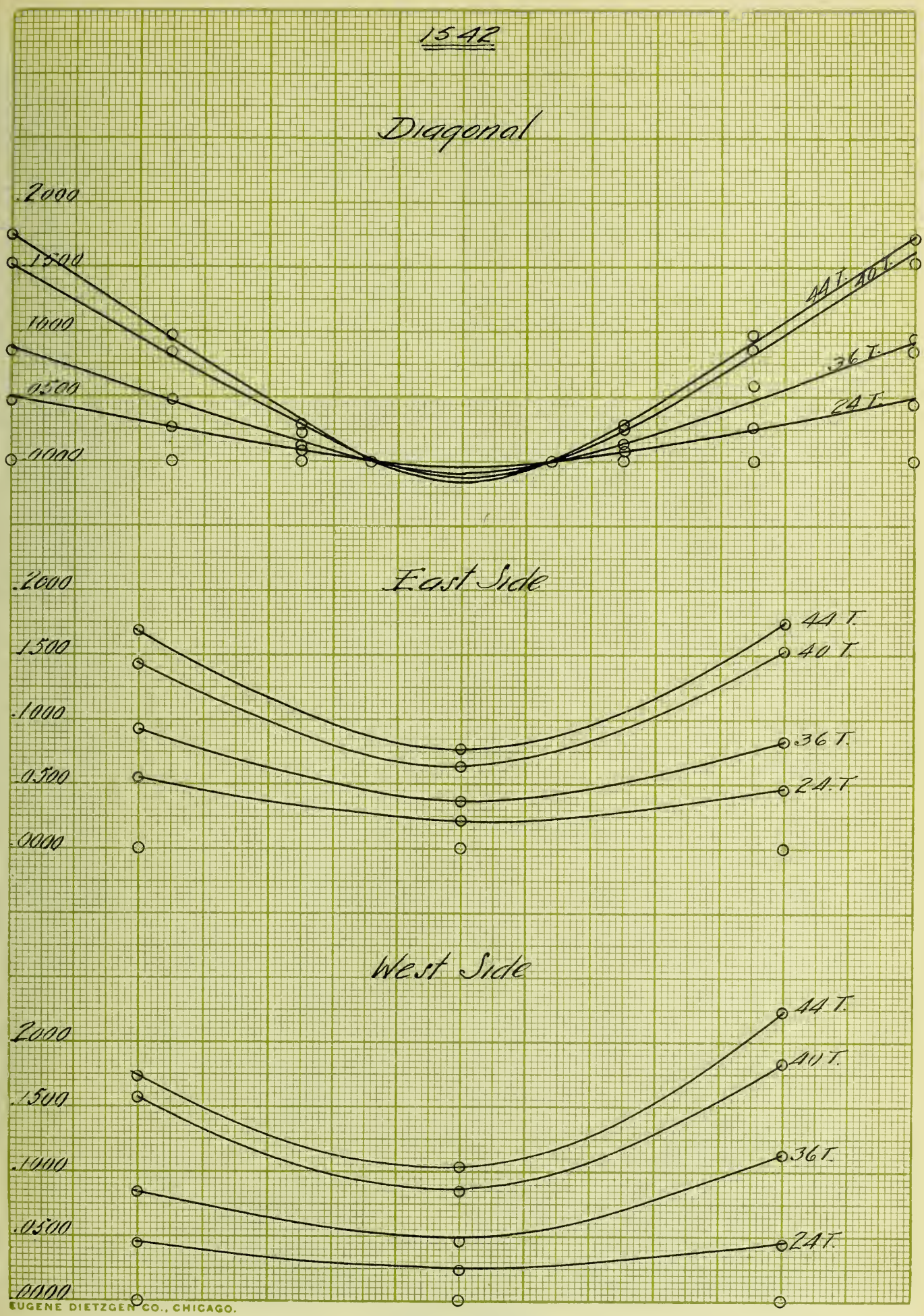
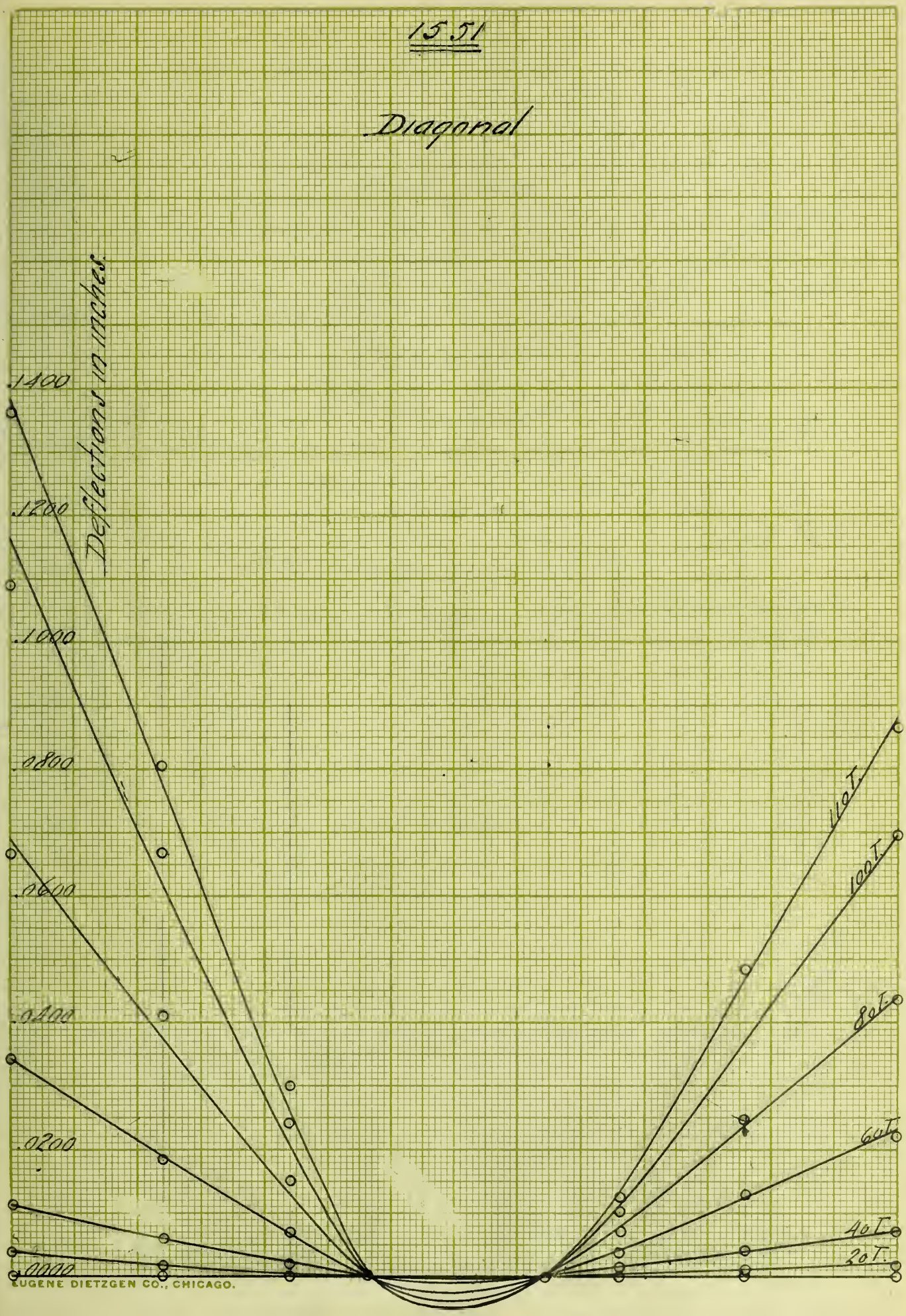
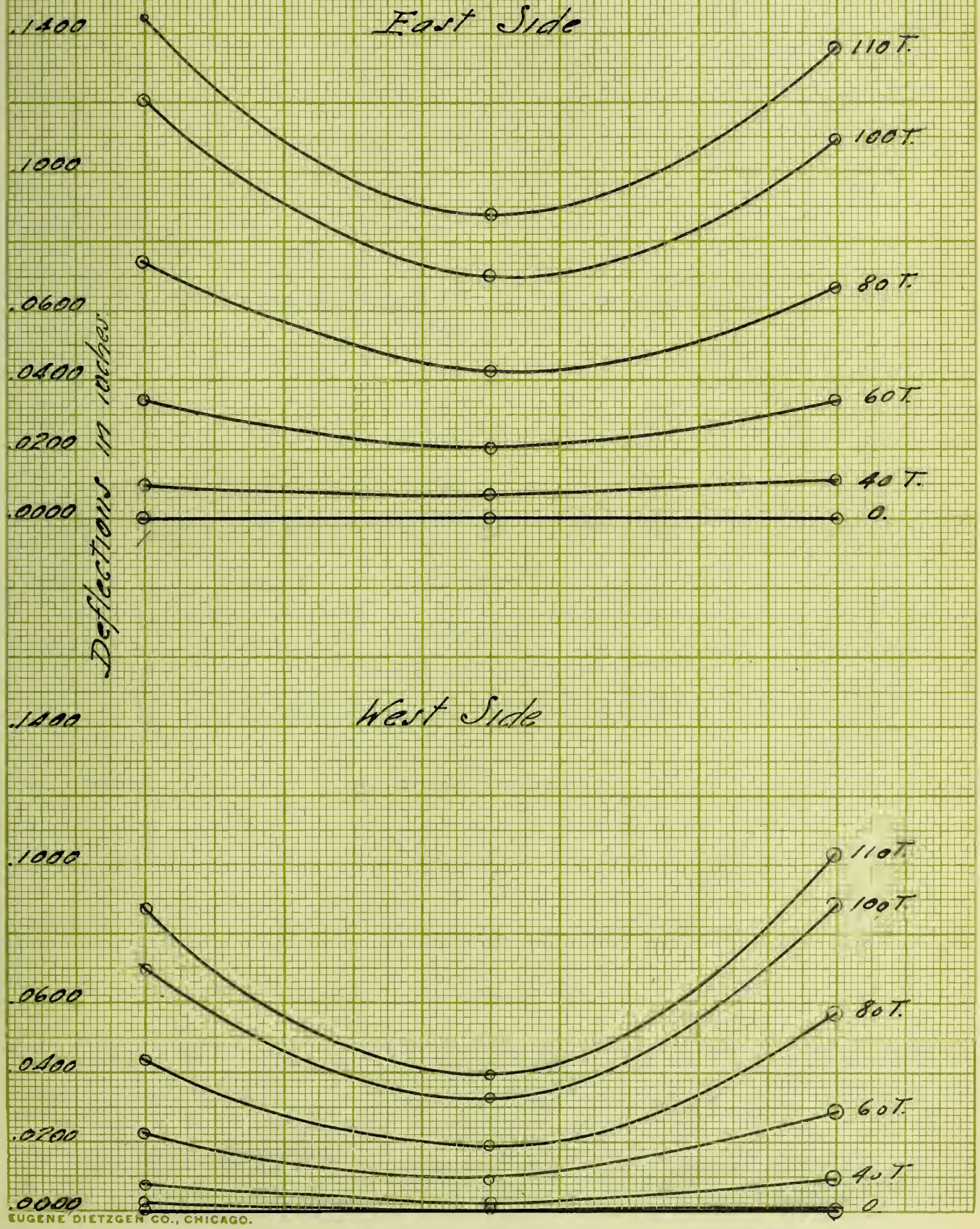

1542
Diagonal
.2000
.1500
.1000
.0500
.0000
44 T.
40 T.
36 T.
24 T.
East Side
.2000
.1500
.1000
.0500
.0000
44 T.
40 T.
36 T.
24 T.
West Side
.2000
.1500
.1000
.0500
.0000
44 T.
40 T.
36 T.
24 T.
EUGENE DIETZGEN CO., CHICAGO.


15.51
Diagonal
Deflections in inches
.1400
.1200
.1000
.0800
.0600
.0400
.0200
.0000
110 T.
100 T.
80 T.
60 T.
40 T.
20 T.
EUGENE DIETZGEN CO., CHICAGO.

1551.



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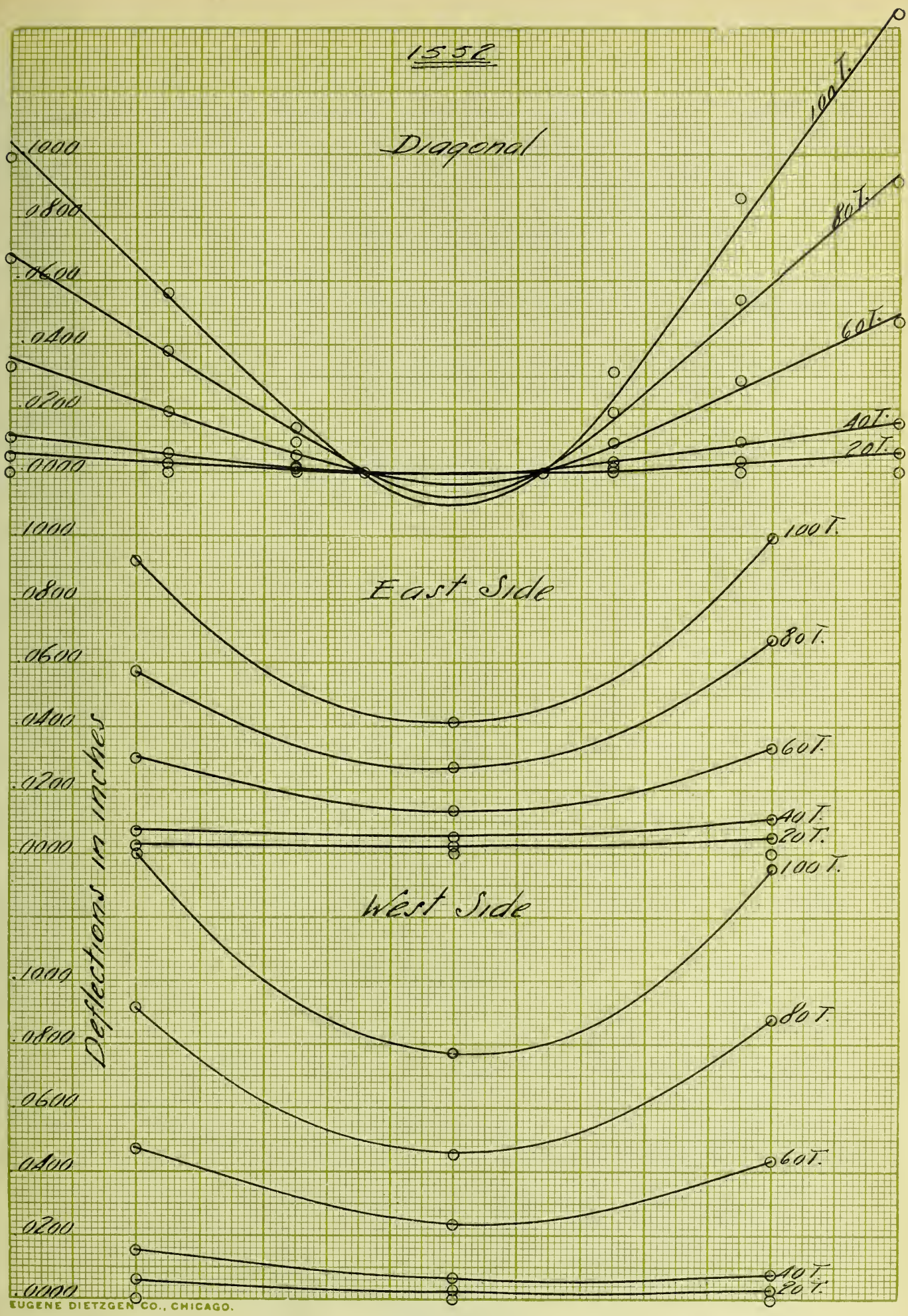

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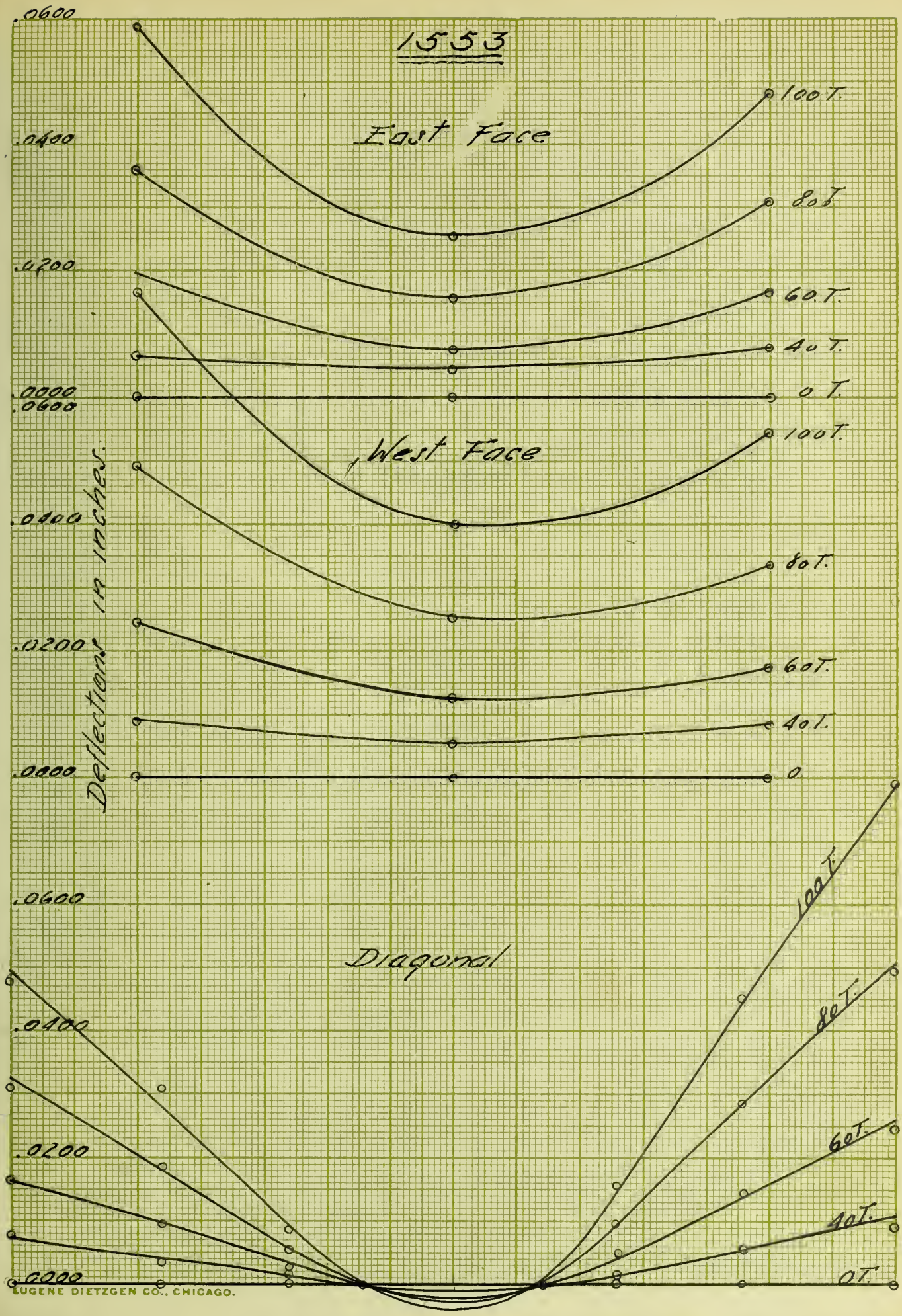
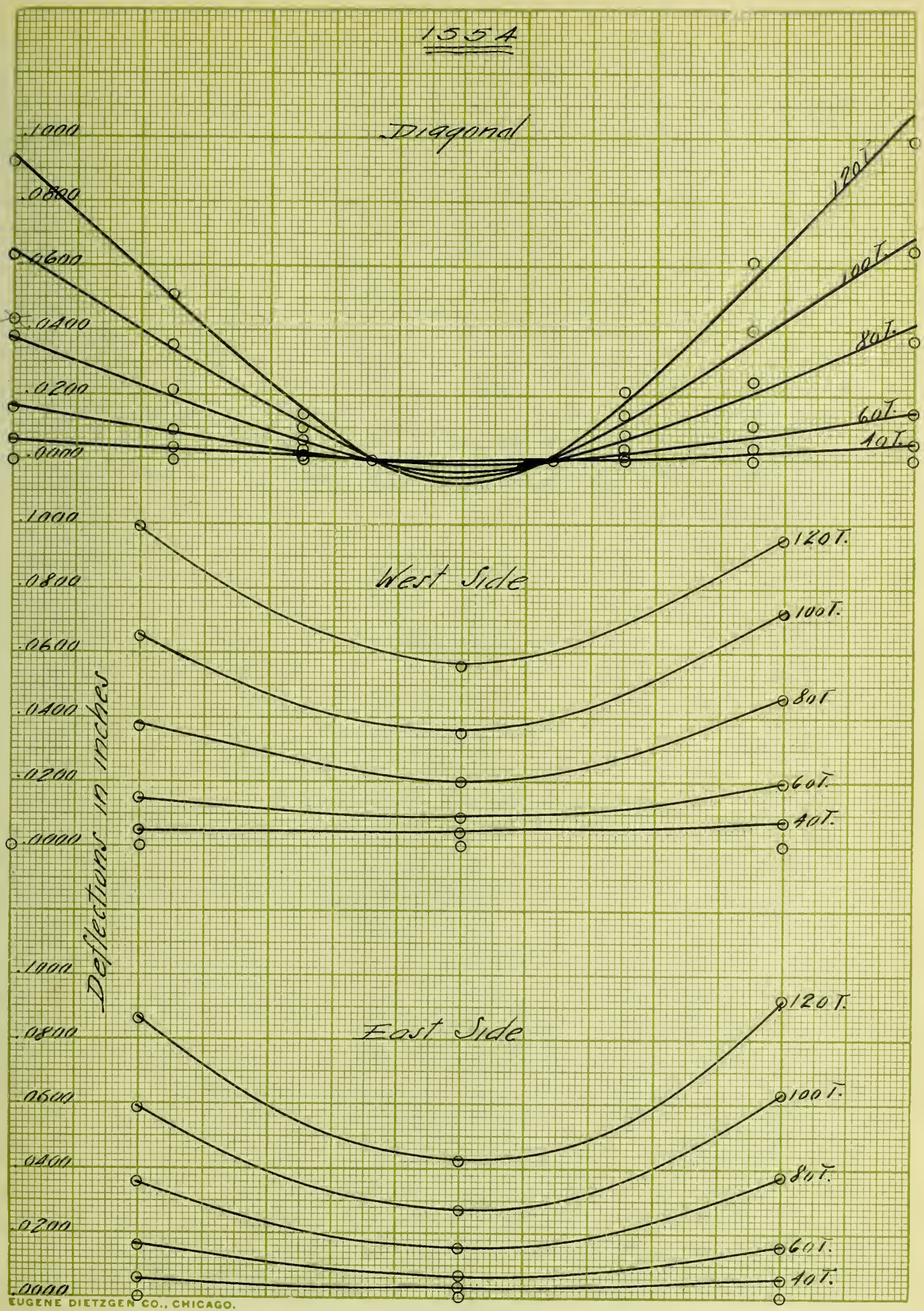

1553
East Face
West Face
Diagonal
Deflection in inches.
.0600
.0400
.0200
.0000
100 T.
80 T.
60 T.
40 T.
0 T.
EUGENE DIETZGEN CO., CHICAGO.



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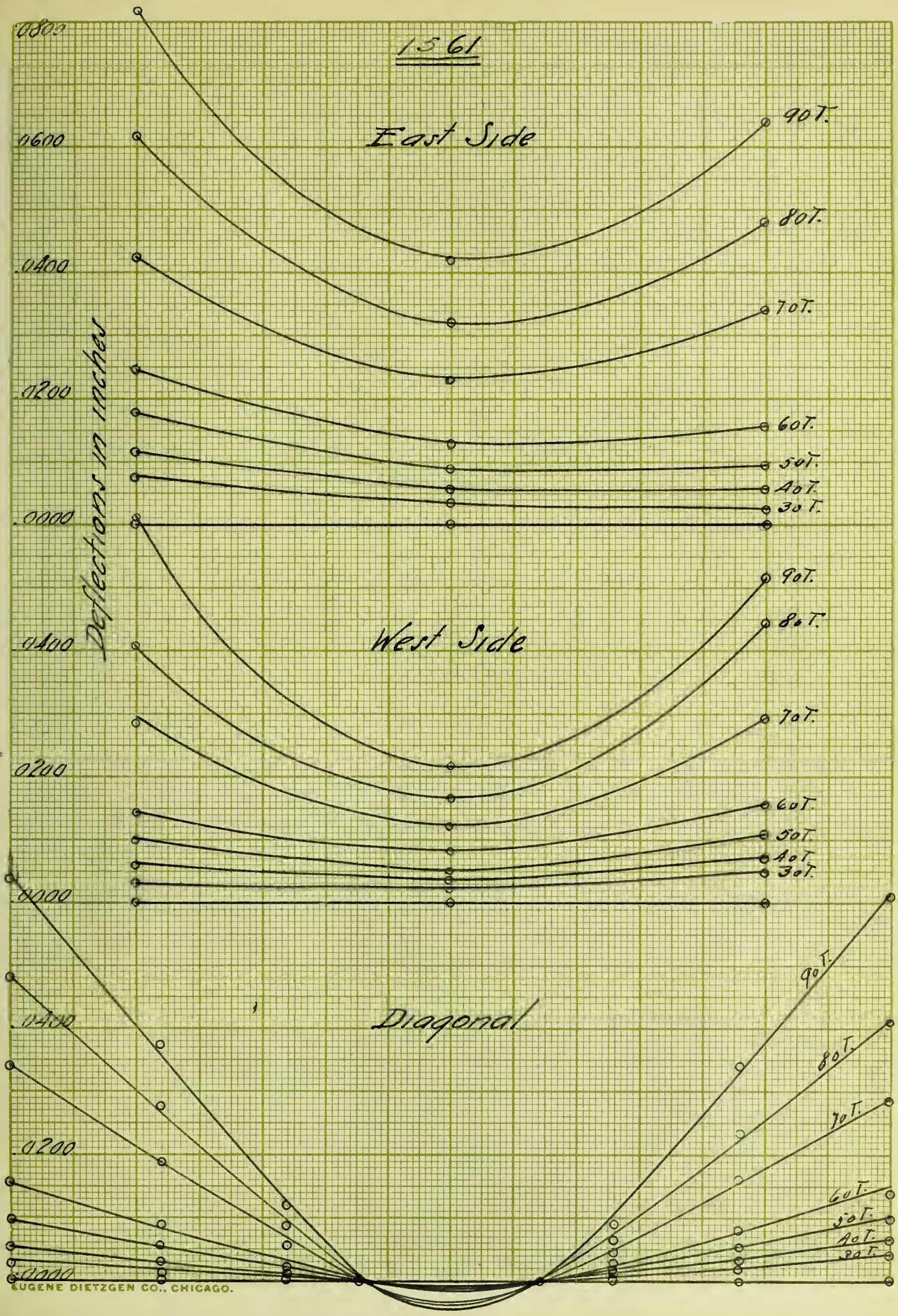
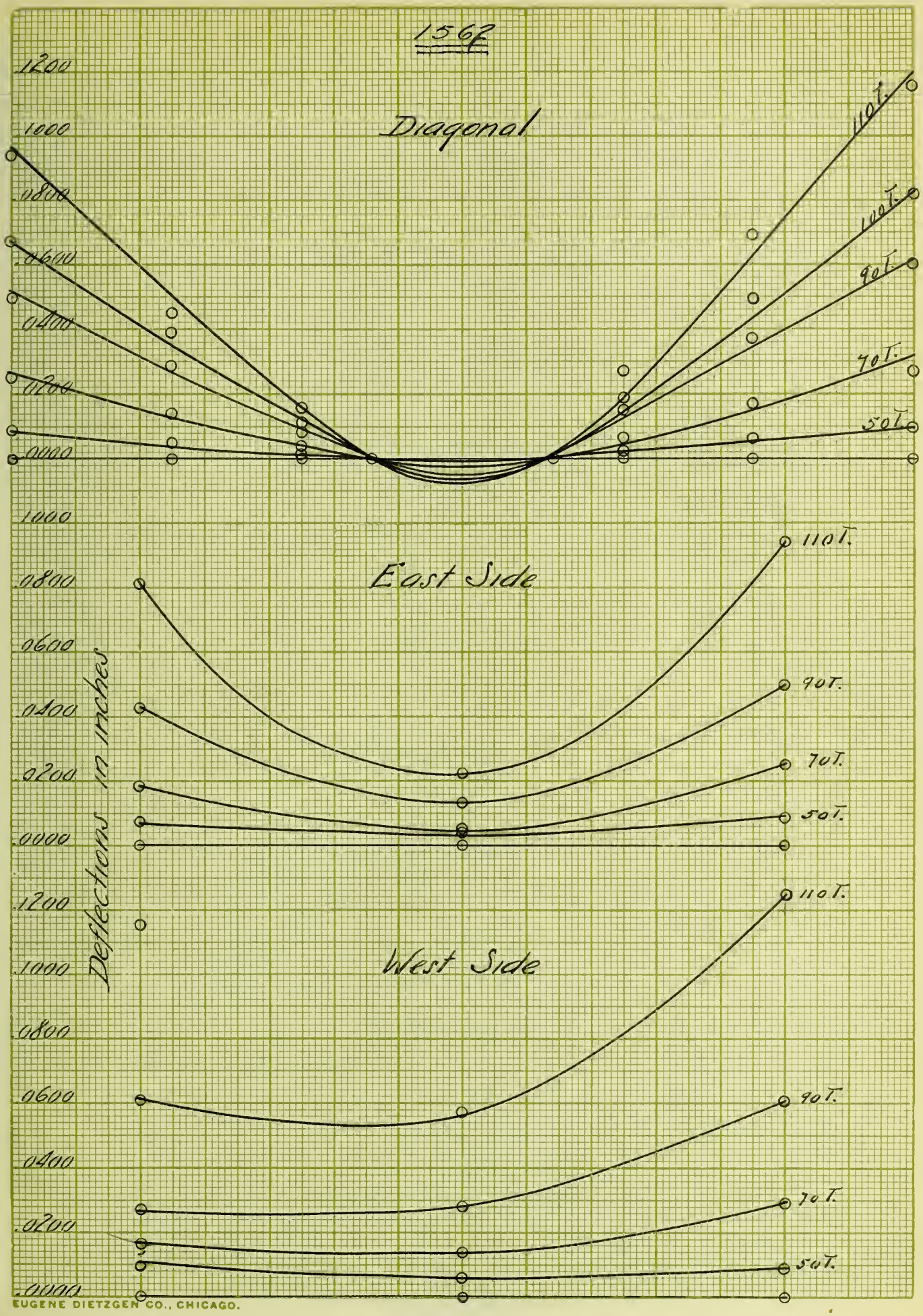

1561
East Side
West Side
Diagonal
Deflections in inches
.0800
.0600
.0400
.0200
.0000
90T.
80T.
70T.
60T.
50T.
40T.
30T.
EUGENE DIETZGEN CO., CHICAGO.



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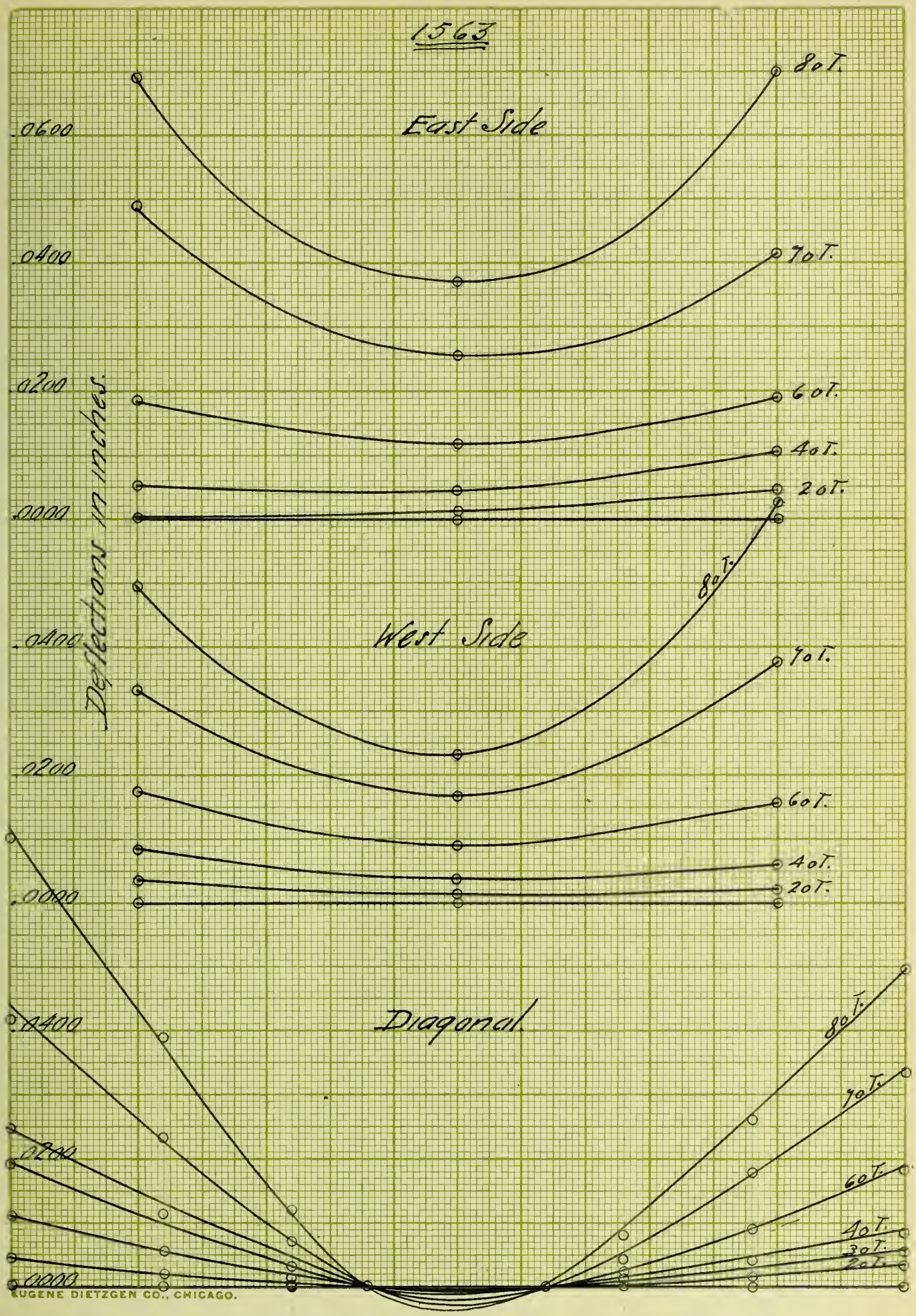
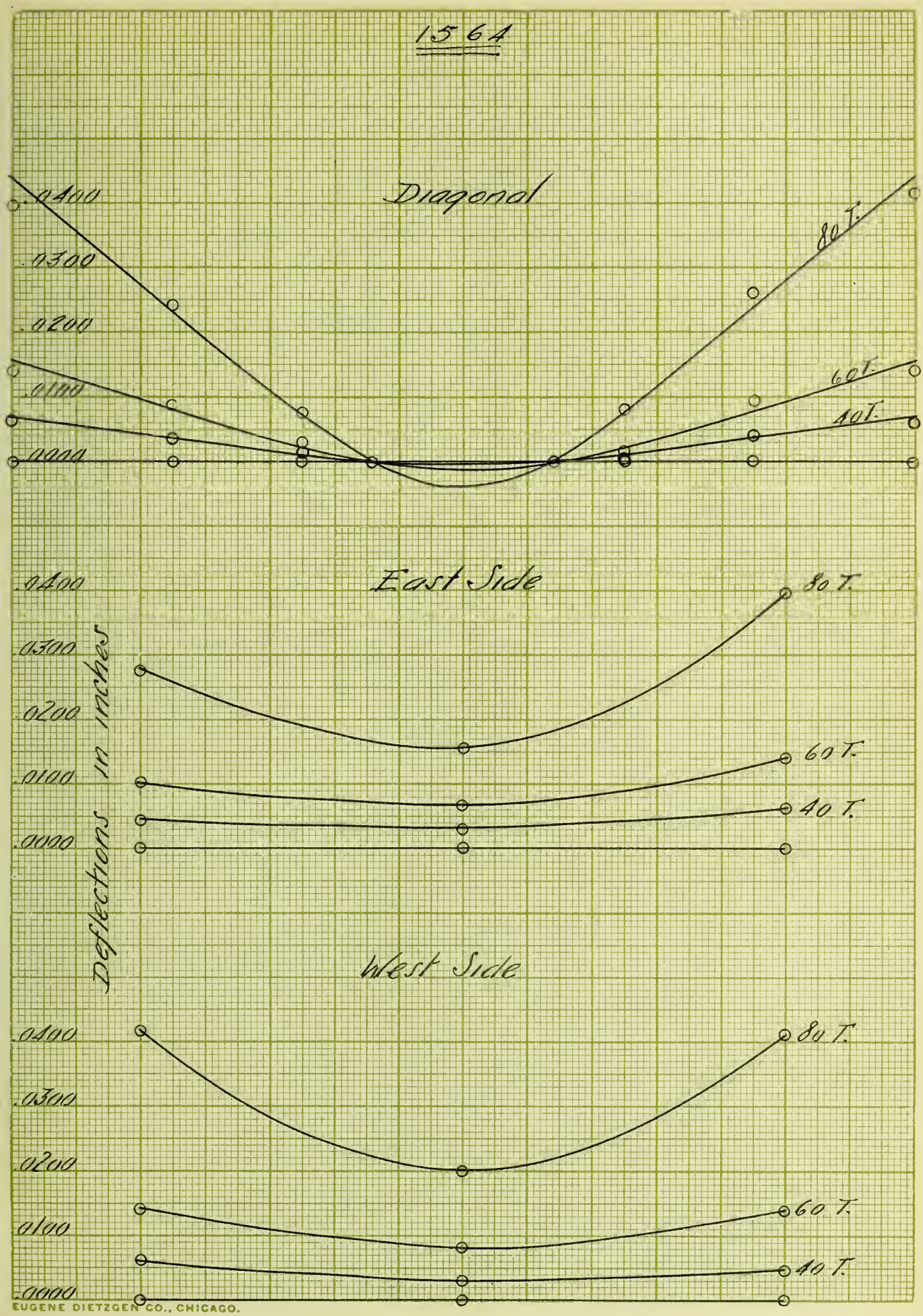

1563
East Side
West Side
Diagonal
Deflections in inches.
.0600
.0400
.0200
.0000
.0400
.0200
.0000
.0400
.0200
.0000
80T.
70T.
60T.
40T.
20T.
80T.
70T.
60T.
40T.
20T.
80T.
70T.
60T.
40T.
30T.
20T.
EUGENE DIETZGEN CO., CHICAGO.



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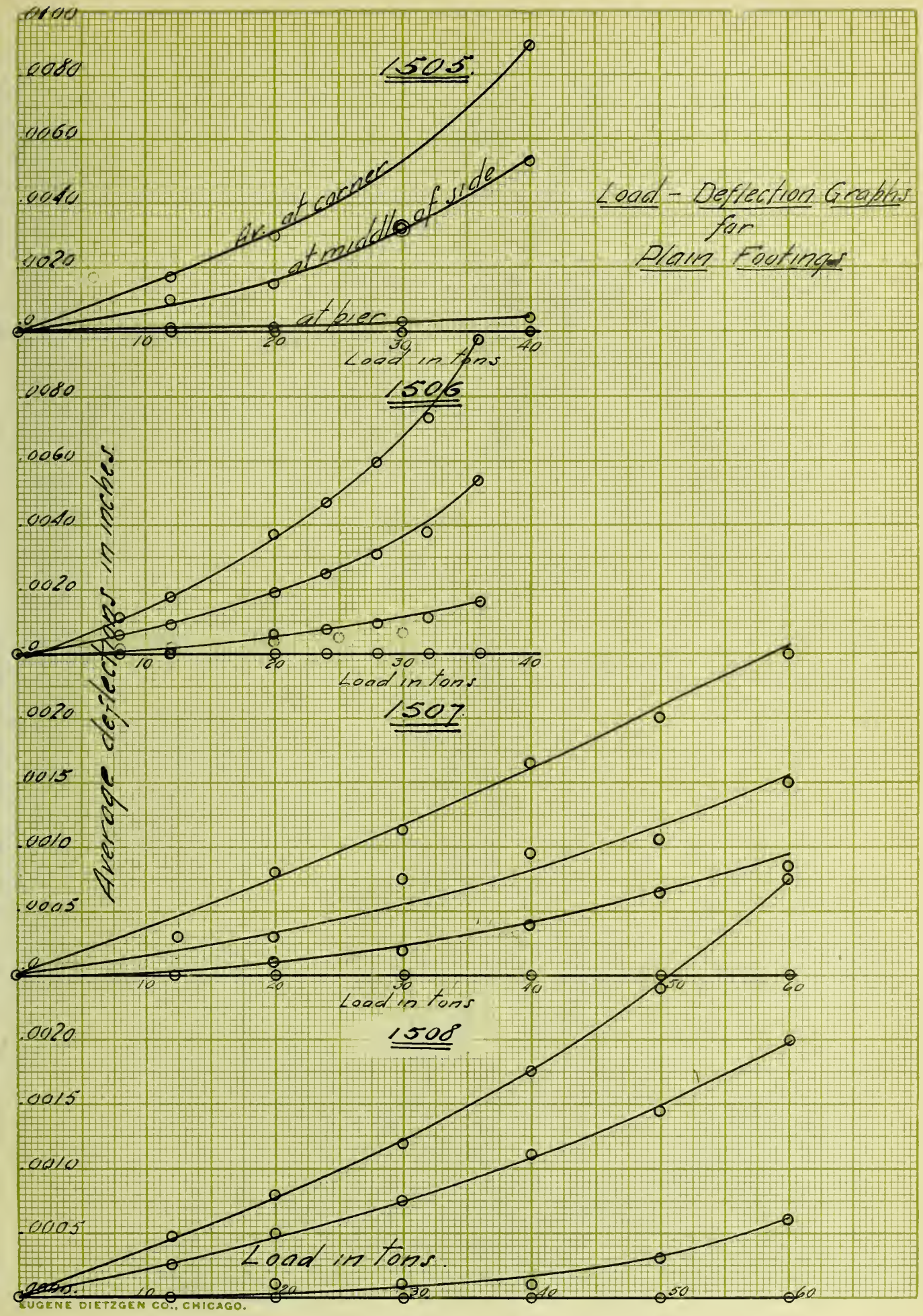

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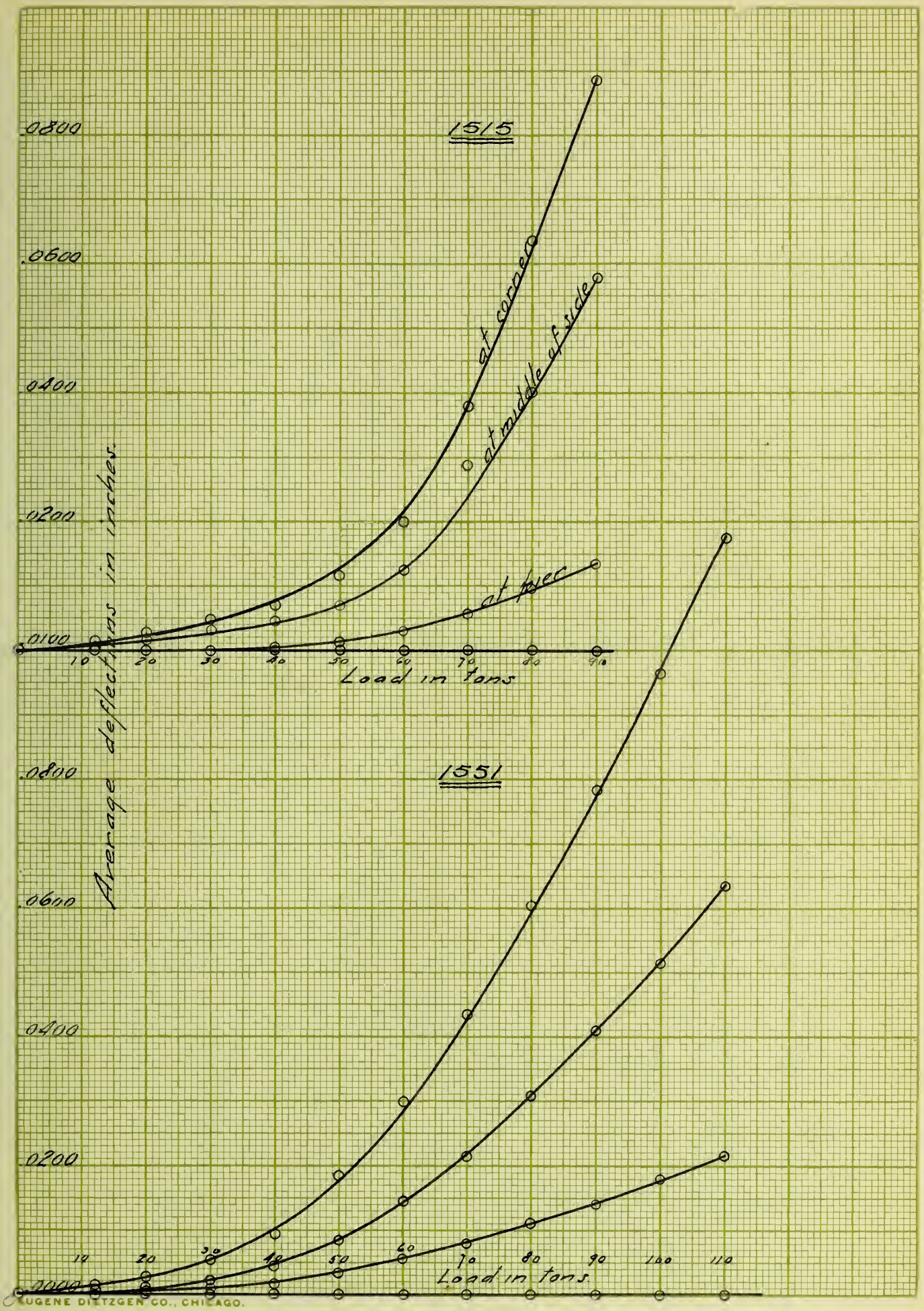

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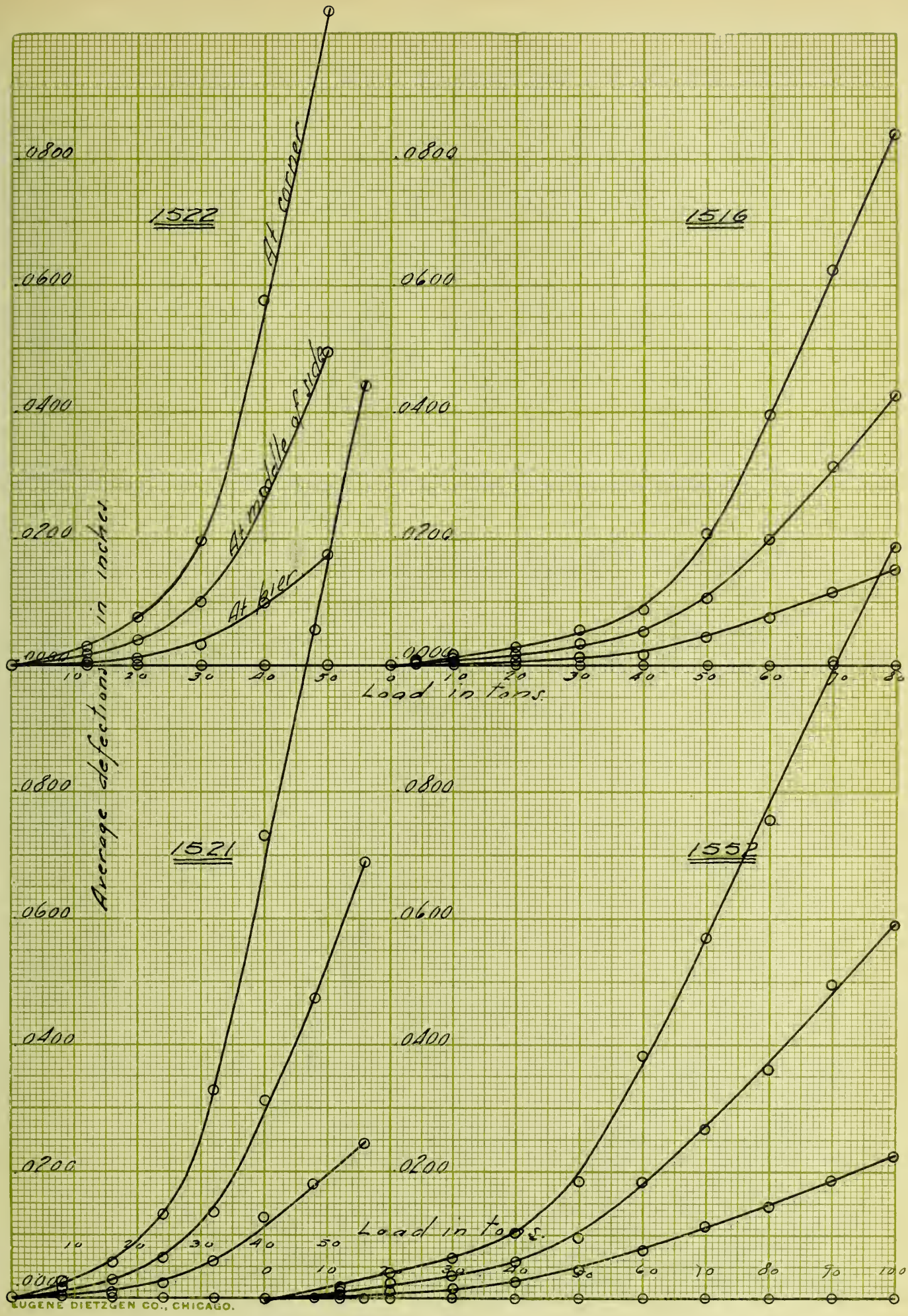
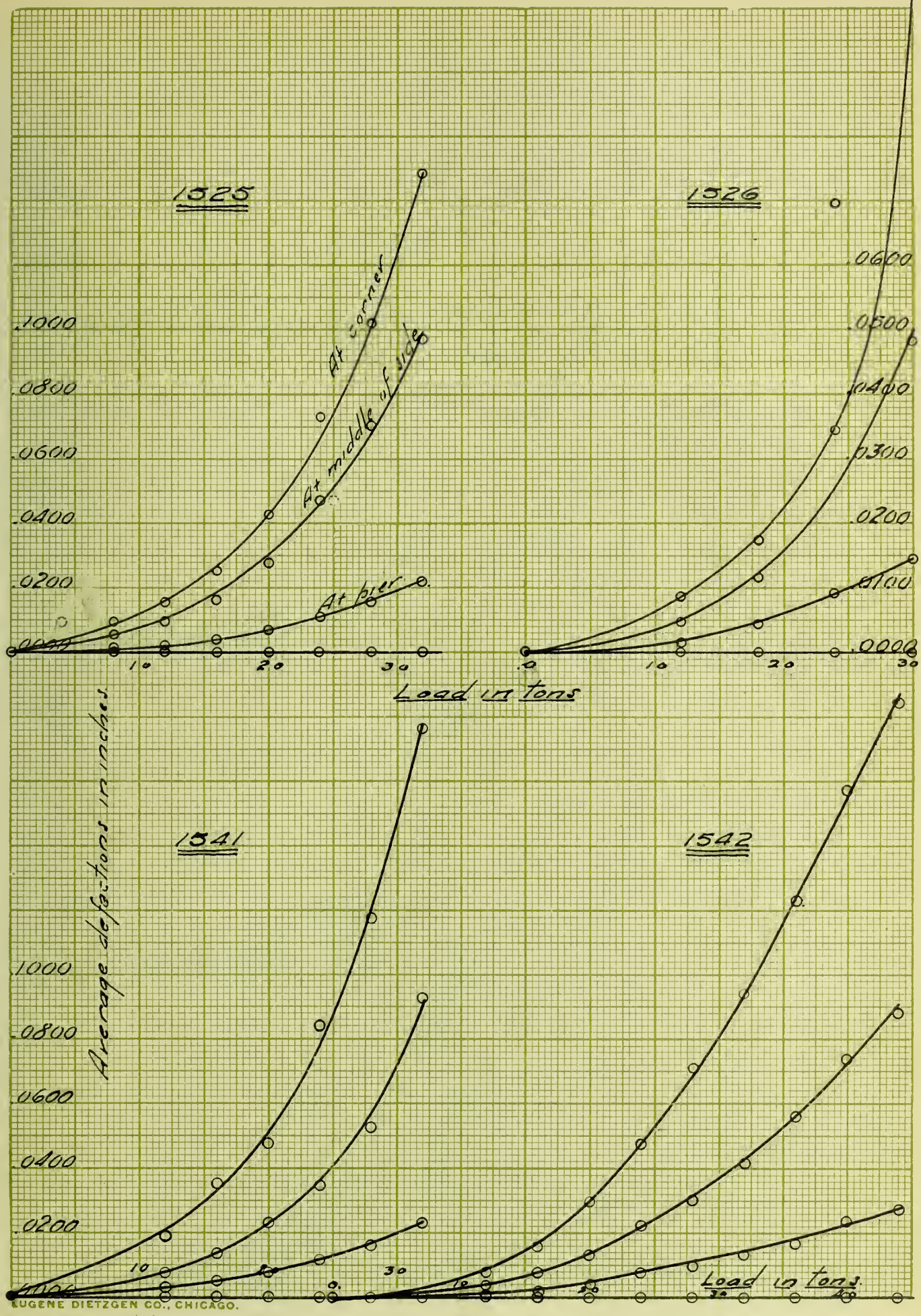
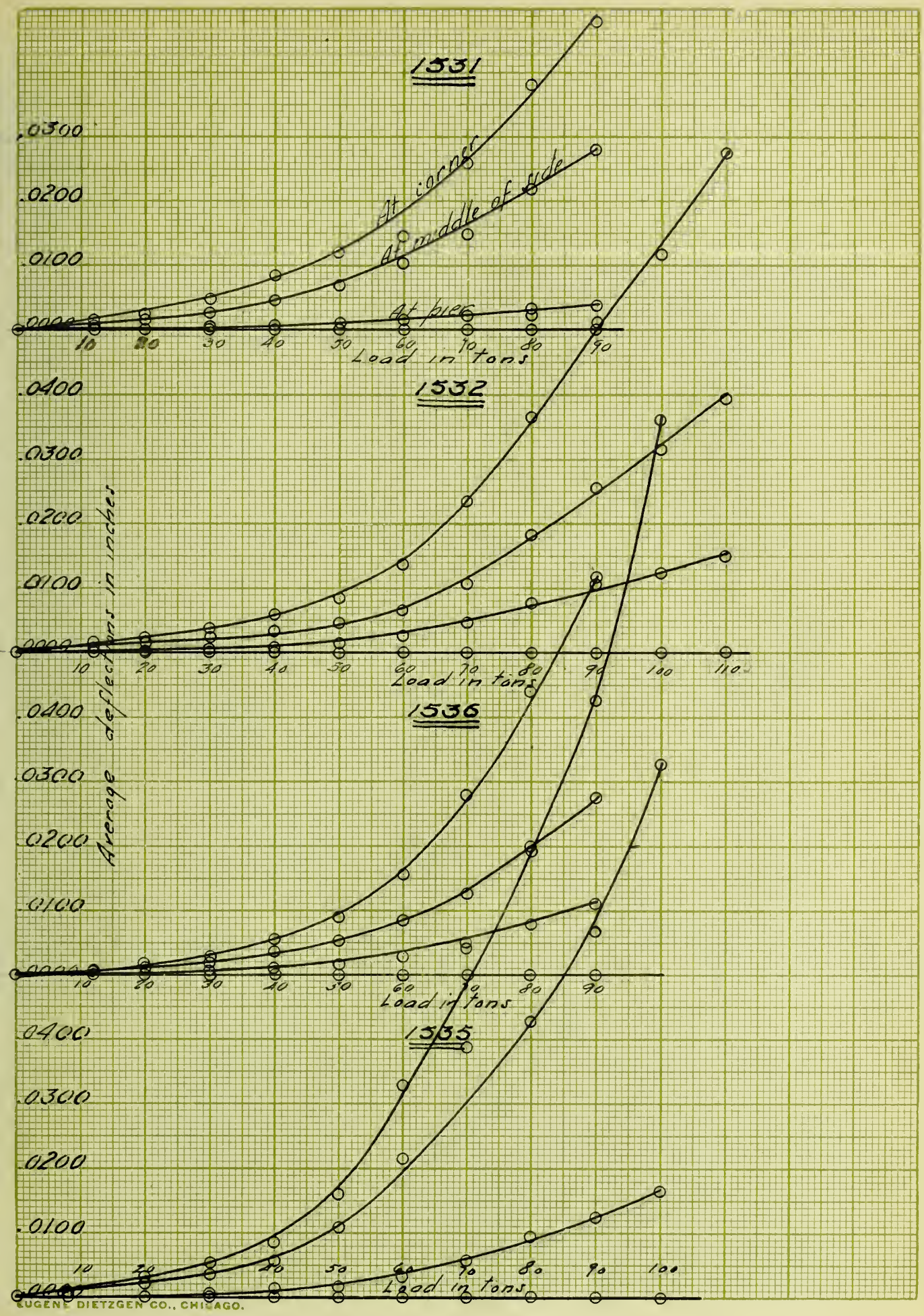

1522
1516
1521
1552
At corner
At middle of span
At pier
Average deflections in inches
Load in tons.
.0800
.0600
.0400
.0200
.0000
10
20
30
40
50
60
70
80
90
100
EUGENE DIETZGEN CO., CHICAGO.


1525
1526
At corner
At middle of side
At pier
.1000
.0800
.0600
.0400
.0200
.0000
.0600
.0500
.0400
.0300
.0200
.0100
.0000
10
20
30
Load in tons
Average deflections in inches
1541
1542
Load in tons.
EUGENE DIETZGEN CO., CHICAGO.



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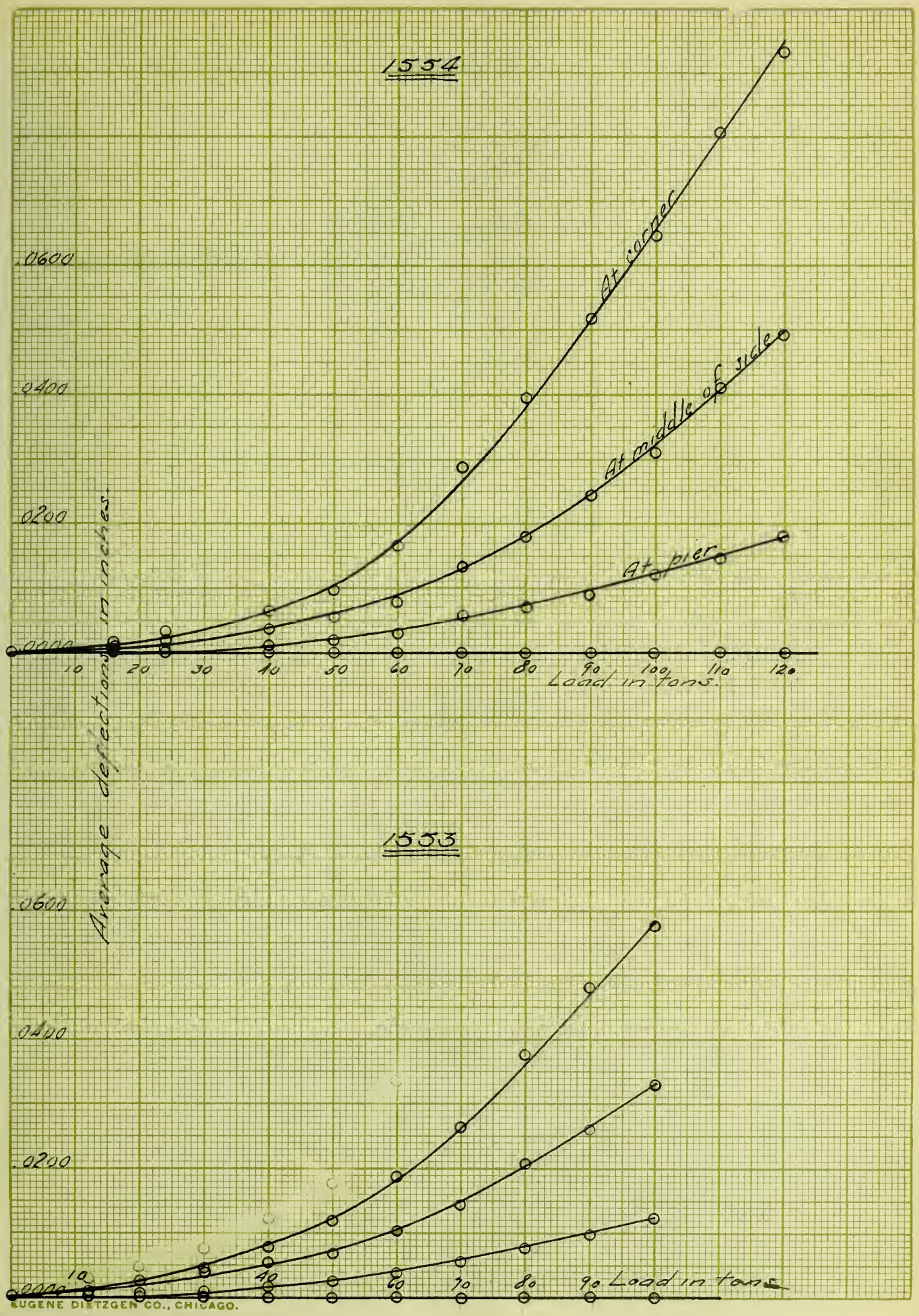
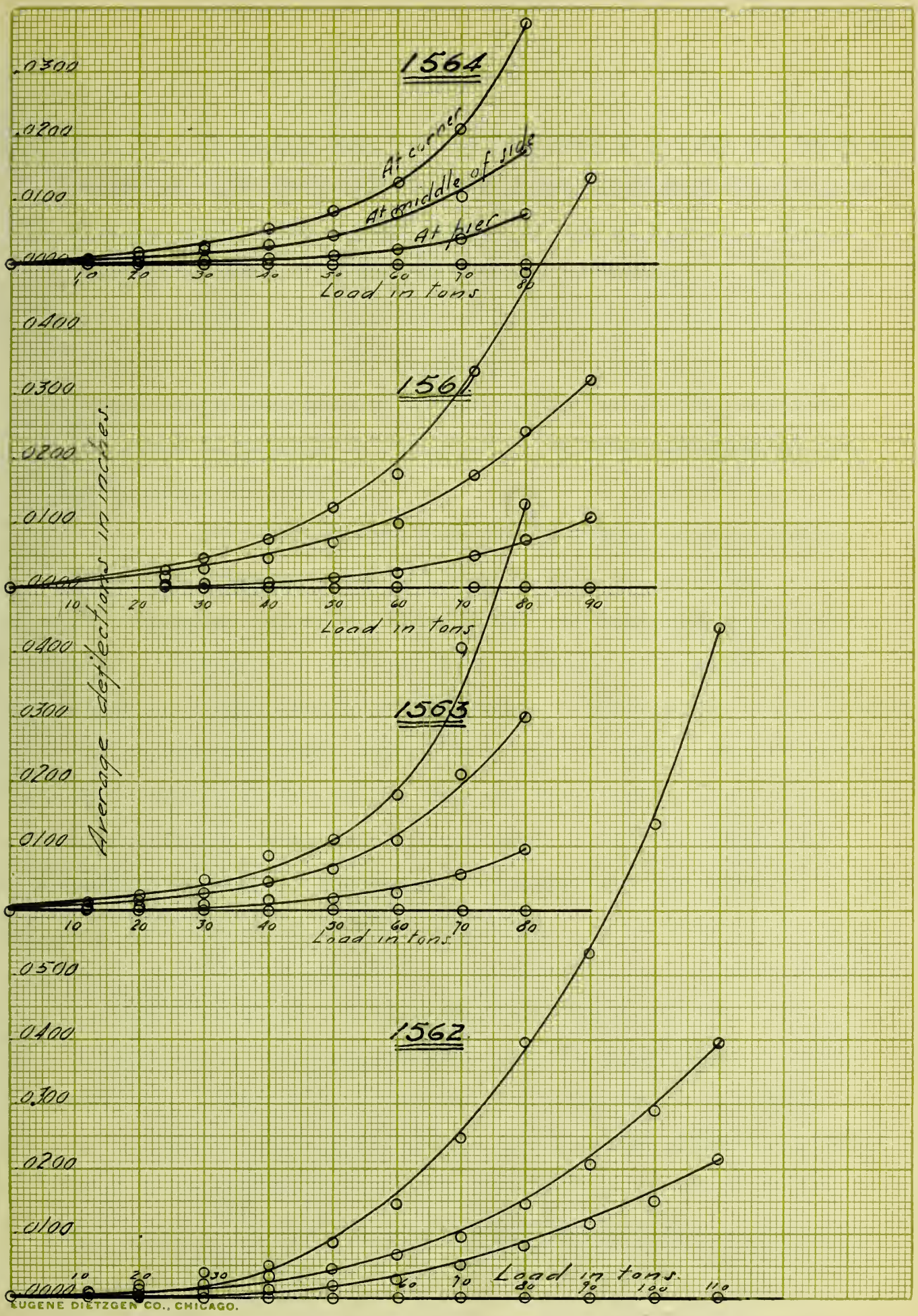

1554
.0600
.0400
.0200
.0000
Average deflections in inches.
At corner
At middle of side
At pier
10
20
30
40
50
60
70
80
90
100
110
120
Load in tons.
1553
.0600
.0400
.0200
.0000
10
40
60
70
80
90
Load in tons
EUGENE DIETZGEN CO., CHICAGO.



EUGENE DIETZGEN CO., CHICAGO.

VI. Discussion.

It is felt that the methods used in making the tests reported in this thesis may be considered to approximate the conditions existing in a pier footing in actual service. The number and scope of the tests are not sufficient to warrant drawing definite conclusions, but a discussion will be given covering some phases of the tests. Of the 28 footings tested, 8 were plain concrete and failed by tension in the concrete, 10 failed in tension in the steel, 6 in shear or diagonal tension, and 4 in bond.

The calculations made in the tables of this thesis have been based upon the use of a portion of the width of the footing as an "effective beam width" for resisting the bending moment produced by one fourth of the load, determined as outlined on pages 3 and 4 of the theory. Examination of the curves for Footing No. 1531, which had a line of deflectometers across the center parallel to the East and West faces, shows that the stress in the steel at the outer edge is not more than one half the stress at the center, assuming that the stresses in this case vary directly as the deflections. If a beam equation is used, considering the steel to be stressed to the yield point for footings which failed by tension, the resulting width determined is in all cases less than the total width of the footing. That this is the case is borne out by the table of calculated data on page 16 and may be shown graphically by Fig. 5. in which the resisting moment is represented by the area ABCD. In the dotted rectangle of width b, the stress in

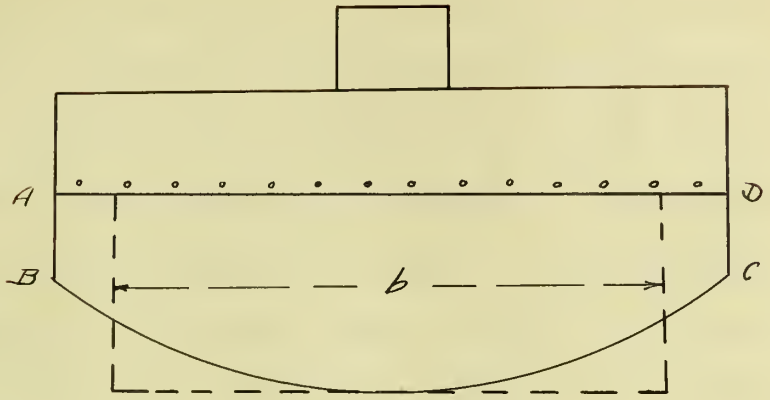


Fig. 5.

the steel is assumed to be equal to the stress in the steel at the center and the area included is equivalent to the area ABCD.

The effective widths in Table 5 are, with a few exceptions, more than two thirds of the total width of the footing. For footings of the dimensions used in these tests, therefore, it is not thought that the economy of varying the spacing of the rods near the edges is enough to warrant the extra labor of so placing them.

Bond failures were characterized by the opening of large cracks in the faces near the corners of the footing and the closing of the hair cracks previously developed near the center of the faces. All of the footings having 3/4-in. rods failed in this manner. The bond stresses shown in Table 5 are quite uniform and about the values that might be expected. The highest bond stresses shown are for the footings which failed in bond. The calculated results, therefore, bear out the assumption that the bond stresses should be calculated for the steel over the width b, determined from the resisting moment.

The results of this series of tests gave values of unit shear varying from 242 lb. per sq. in. to 531 lb. per sq. in.,

computed as described on page 6. These values compare favorably with the values of ultimate unit shear in reinforced concrete of about the same composition, as given in Turneaure and Maurer's "Principals of Reinforced Concrete Construction". Hence the tests warrant the assumptions in regard to the area resisting shear stated under "Theory" on page 6 , when checking for unit shear in footings that have been designed for resisting moment.

A comparison of Footings No. 1531 and 1532 with 1561, 1562, 1563, and 1564 shows that there is no economy in placing steel in four directions. Footings No. 1531 and 1532 sustained an average ultimate load of 286000 lb. and failed in shear, while the highest load carried by any of the others having the same amount of steel in four directions is 260000 lb. and the average of the four having the reinforcement in four directions is 228000 lb., all failing in tension. The differences in the averages of the ultimate loads of 286000 lb. in one case and 228000 lb. in the other is 58000 lb. This would probably have been greater had the strength of the footings reinforced in two directions depended upon the steel and not upon the shear or diagonal tension. When the rods are placed in four directions, there seems to be no advantage in omitting the short diagonal rods in the corners for in each case where this was done, the footing failed at a smaller ultimate load. The difference in the average ultimate loads for the two pairs of footings constructed for this comparison is 36000 lb.

In the plain footings the same disposition of the forces and the same plane of maximum stresses due to moment and shear were assumed. The effective widths given in Table 3 vary from

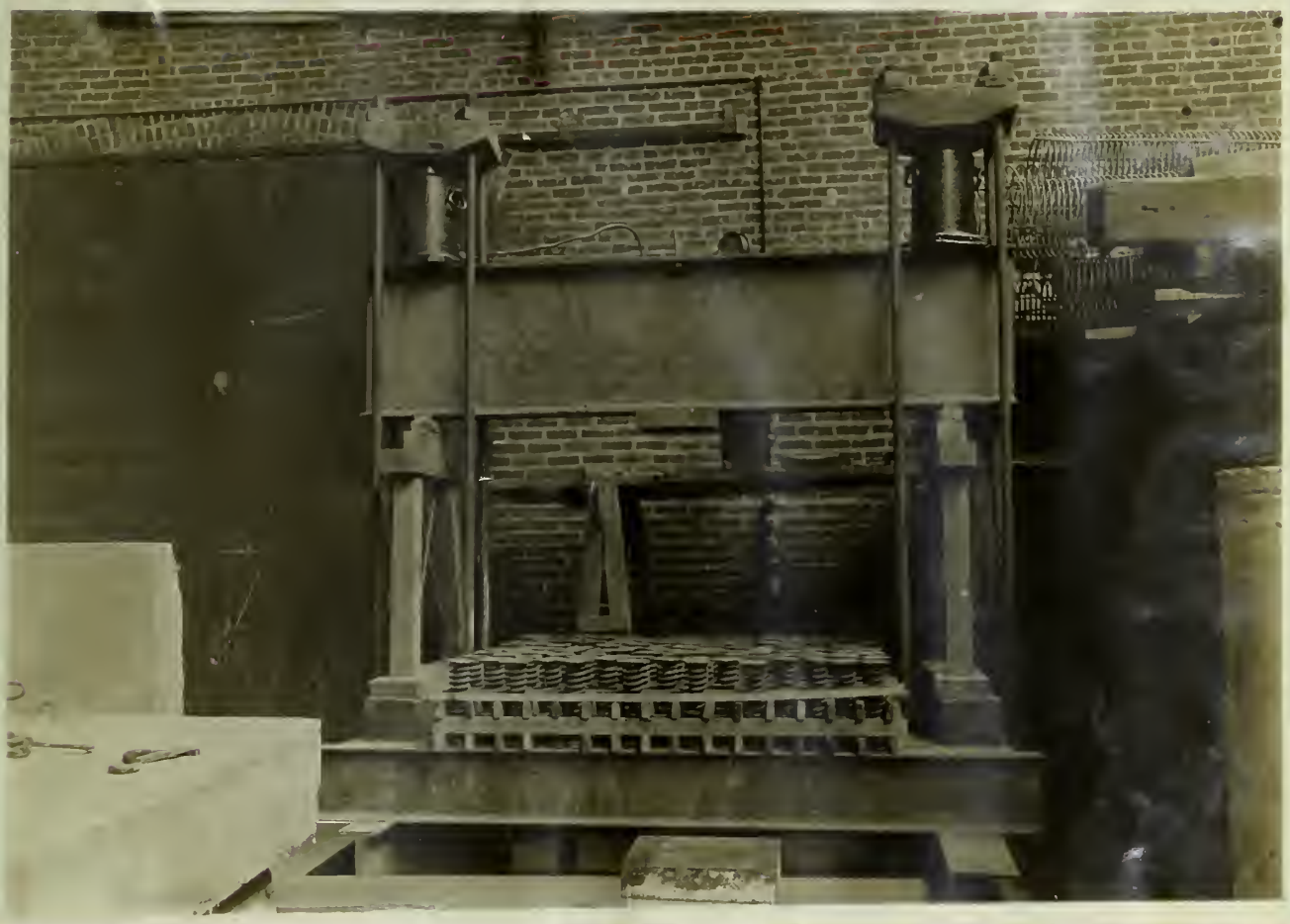
32.6 in. to 63.6 in. The largest value is for one of the footings 6 in. deep and the smallest is for one of the footings 12 in. deep. As far as can be determined from the tests on plain footings the effective width seems to be no function of the depth. The values given in the last column in Table 3 where this width is expressed in terms of a constant plus a function of the depth, further bear out this conclusion.

Owing to the fact that the effective width varies as well as the depth, the ultimate strength of the footings does not vary as the square of the depth. Supposing, for example, that the effective width decreases as the depth of the footing increases, as the values in Table 3 would seem to indicate, then the strength is not increased four times when the depth is doubled.

From the set of curves showing the exaggerated shape of various lines across the footings under load, it may be noticed that the diagonal from the corner of the pier to the corner of the footing is, in nearly all cases, a straight line and in some cases shows a slight crown. Since the line through the center in all cases shows a curvature concave from the top, it is natural to expect that if the footing were round instead of square, any section would show some upward curvature, i.e., concave from the top. The direction of the reinforcing can have very little to do with it since the same thing was apparent in the footings having no reinforcement. The exact nature of the internal stresses which cause this phenomenon cannot, of course, be determined from these tests, but it is evident that the action is due to the shape of the footing tested.

It might also be interesting to note in connection

with the plain footings that appreciable deflections amounting to about .0035 of an inch were obtained in the footings 18 in. deep at about one half of the ultimate load. The deflections were always a maximum at the corners as may be seen by reference to the deflection graphs.



General view of testing apparatus showing bed of 113 springs on foundation of 10 inch I-beams. The arrangement of the hydraulic jacks and pumps is also shown. The small sticks were used simply to space the springs and hold them in an upright position until the footing was mounted upon them.

View showing wreck of the apparatus after the violent failure of one of the eighteen-inch plain footings. The break in this case was almost directly in the center of the footing north and south leaving the column stub hanging by two of the rods with which it was reinforced. In the foreground may be seen some of the I-beams of the foundation that have been turned over on their sides by the force of the failure.

View showing the other eighteen-inch deep plain footing after faiure. The footing in this case broke into three pieces and was not as diastrous to the apparatus as its mate, 1508. The column stub was in this case entirely separated from the other parts of the footing except for the reinforcing rods in the corners.

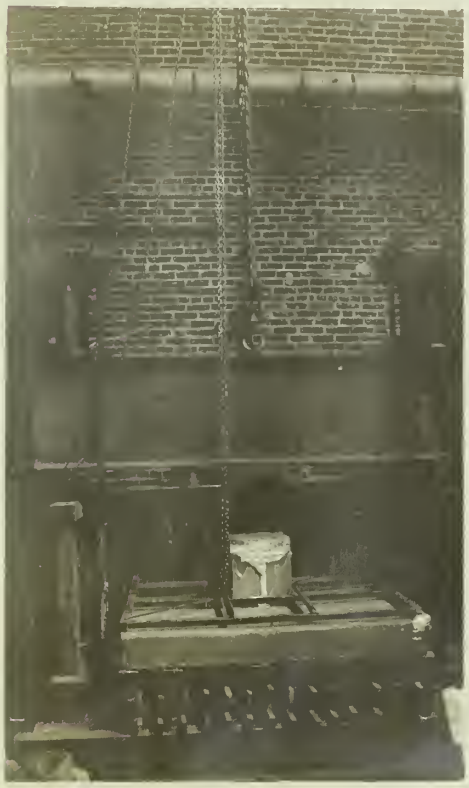
Footing No. 1521 which failed gradually in tension and finally punched in as shown. This footing had 0.58% steel at a depth of seven inches.

Footing No. 1551 after failure. The footing failed by the punching through of the column stub causing most of the concrete below the steel to drop off. This is evidently a failure in the shearing strength of the concrete as the small hair cracks in the faces that had appeared first at 100 tons closed entirely at failure. See photograph on next page.

The two pumps used on the hydraulic jacks can be seen mounted on their platform back against the wall.

Under side of 1551 showing a shear failure with corrugated bar reinforcement. The line of shear is practically 45 degrees as was assumed in the calculations of the shearing stresses developed. See photograph on preceeding page.

Frame with Ames deflectometers mounted and ready for the load.



One of the small reinforced footings after the instruments have been removed from the frame.

Part of 1552 showing manner of shear failure The reinforcing bars exposed are 9/~~16~~ ½ in. corr. ~~rounds~~.

Under side of 1531 showing another shear failure. In this case the column stub and all the steelwere separated from the remainder of the footing.

Arrangement of bed of 113 springs.

Arrangement of bed of 225 springs.
This number was used only for some of the heavy footings that could not safely be loaded to failure on 113 springs.

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